NELSON GEOTECHNICAL ASSOCIATES, INC.
GEOTECHNICAL ENGINEERS & GEOLOGISTS

Main Office
17311 – 135th Ave NE, A-500
Woodinville, WA 98072
(425) 486-1669 · FAX (425) 481-2510

Engineering-Geology Branch
5526 Industry Lane, #2
East Wenatchee, WA 98802
(509) 665-7696 · FAX (509) 665-7692

September 28, 2016

Mr. J.J. Engler
Seacon, LLC
165 NE Juniper Street
Issaquah, Washington 98027

Geotechnical Engineering Evaluation
Union Hill Self-Storage
Union Hill Lot 4 – NE 76th Street
Redmond, Washington
NGA Project No. 969616

Dear Mr. Engler:

We are pleased to submit the attached report titled "Geotechnical Engineering Evaluation – Union Hill Self Storage – Union Hill Lot 4 – NE 76th Street - Redmond, Washington." This report summarizes the existing surface and subsurface conditions within the site and provides recommendations for the proposed site development. Our services were completed in general accordance with the proposal signed by you on August 25, 2016

The site is located along the southern side of NE 76th Street immediately east of the property located 18459 NE 76th Street. The parcel number for the property is known as 20120906900004. The site is a roughly rectangular shaped parcel covering approximately 3.02 acres. The site is currently undeveloped. We understand that the proposed development plans include construction of a three-story self-storage building and associated parking within the southern portion of the site along with a lower parking lot area within the northern portion of the property along NE 76th Street. Tiered retaining walls are proposed around the western and northern portions of the property to bring the site grades up to finished grade elevations. The retaining walls will likely consist of the Hilfiker Welded Wire system and reinforced Keystone walls. The proposed building finished floor elevation is approximately 90 feet +/- while the finished elevation of the lower parking lot is proposed to be approximately 74 to 76 feet +/-. The lowest portion of the site along the western property boundary is approximately 55 feet. Specific grading plans were not available at the time this report was prepared. However, we do anticipate significant grading activities being performed within the site to construct the retaining walls and to bring the site to the proposed elevations. Specific stormwater plans were also not available at the time this report was prepared. However, we anticipate that due to the relatively silty nature and thickness of the fill soils that underlie the surface of the site that infiltration is likely not feasible and that stormwater will likely be directed to an appropriate stormwater collection system within the site.

We explored the subsurface soil and groundwater conditions on August 31, 2016 with eleven trackhoe-excavated test pits. In general, the test pits exposed silty sand with gravel with varying amounts of debris to the depths explored. We interpreted the soils to be undocumented fill soils that were placed here as a part of previous grading and filling performed within the property. Review of a previously prepared geotechnical report for the property indicated that the eastern portion of the property was explored with nine drilled borings extending to depths in the range of 26.5 to 46.5 feet below the existing ground surface. These borings generally encountered undocumented fill soils consisting of lean clay, clayey sand, sandy silt, silty sand, and silty gravel with varying amounts of cobbles, boulders, organics, and wood debris within the upper portion of the borings. Seven of the nine borings were completed within the fill soils. Within the two northern borings, sands and gravels interpreted to be native recessional outwash were encountered at approximately 40 feet below the existing ground surface or an elevation of 45 feet.

We have concluded that the site is generally compatible with the planned development from a geotechnical standpoint. We understand that the proposed building will likely be supported on both newly placed structural fill associated within the retaining wall construction on the western portion of the property and the existing undocumented fill soils within the eastern portion of the property. To minimize the potential for foundation settlement and other settlement-related problems, we recommend that that the portion of the building foundations located within the existing undocumented fill soils within the eastern portion of the site be overexcavated by a minimum of four feet and the overexcavation be backfilled with two, two-foot thick layers of crushed rock fully wrapped with a Tensar TX160 geogrid up to the foundation subgrade. The eastern portion of the building outside of the newly placed structural fill associated with the retaining walls within the western portion of the property should be directly supported on the crushed rock wraps. Foundations within the newly placed structural fill along the western portion of the property could be directly on the newly placed structural fill.

Alternatively, the structure foundation loads could be supported on modified ground in the form of compacted rock aggregate piers. This type of ground modification includes augered or vibratory excavations within the foundation and slab areas extending through the upper loose/soft soils. Compacted crushed rock fill is placed within the excavations to further densify the upper loose/soft soils and reducing the potential for future settlement within the building elements. The building foundations and slabs are then supported directly on the compacted rock aggregate piers.

If settlement of the slab on grade and future slab maintenance can be tolerated, the undocumented fill material found within the slab area should be over-excavated approximately two feet and replaced with crushed rock. If settlement of the slab on grade and future slab maintenance cannot be tolerated, the slab should be supported on the geogrid wraps or the compacted rock aggregate piers. If the slab is supported on aggregate piers, the slab should be designed as a structural element, and sized and reinforced accordingly. If the slab is supported on a crushed rock layer, added rebar and cold joints should be incorporated into the slab design to reinforce the slab and minimize damage caused by potential future settlement. In the attached report, we have also included general recommendations for site grading, retaining walls, pavement subgrade and drainage.

It has been a pleasure to provide service to you on this project. Please contact us if you have any questions regarding this report or require further information.

Sincerely,

NELSON GEOTECHNICAL ASSOCIATES, INC.

Khaled M. Shawish, PE
Principal

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INTRODUCTION

This report presents the results of our geotechnical engineering investigation and evaluation of the Union Hill Self-Storage project in Redmond, Washington. The project site is known as Lot 4 Union Hill located along the southern side of NE 76th Street, as shown on the Vicinity Map in Figure 1. The purpose of this study is to explore and characterize the site's surface and subsurface conditions and to provide geotechnical recommendations for the proposed site development.

We understand that the proposed development plans include construction of a three-story self-storage building and associated parking within the southern portion of the site along with a lower parking lot area within the northern portion of the property along NE 76th Street. Tiered retaining walls are proposed around the western and northern portions of the property to bring the site grades up to finished grade elevations. The retaining walls will likely consist of the Hilfiker Welded Wire system. The proposed building finished floor elevation is approximately 90 feet +/- while the finished elevation of the lower parking lot is proposed to be approximately 74 to 76 feet +/-. The lowest portion of the site along the western property boundary is approximately 55 feet. Specific grading plans were not available at the time this report was prepared. However, we do anticipate significant grading activities being performed within the site to construct the retaining walls and to bring the site to the proposed elevations. Specific stormwater plans were also not available at the time this report was prepared. However, we anticipate that due to the relatively silty nature and thickness of the fill soils that underlie the surface of the site that infiltration is likely not feasible and that stormwater will likely be directed to an appropriate stormwater collection system within the site. The existing conditions and proposed improvements are shown on the Site Plan in Figure 2.

SCOPE

The purpose of this study is to explore and characterize the site surface and subsurface conditions, and provide general recommendations for site development. Specifically, our scope of services includes the following:

1. Review available soil and geologic maps of the area, available plans, and any available geotechnical reports for the property.
2. Explore the subsurface soil and groundwater conditions within the proposed retaining wall alignments with trackhoe excavated test pits. Trackhoe was subcontracted by NGA.
3. Perform laboratory classification and analyses on soil samples obtained from the explorations, as necessary.
4. Provide recommendations for site grading and earthwork, including structural fill.
5. Provide recommendations for foundation support and slab-on-grade subgrade preparation.
6. Provide recommendations for temporary and permanent slopes.
7. Provide recommendations for retaining walls.
8. Provide recommendations for pavement subgrade.
9. Provide recommendations for site drainage and erosion control.
10. Provide recommendations for Hilfiker Welded-Wire retaining wall design and construction.
11. Provide calculations and engineering details for planned fill and Hilfiker Retaining Walls.
12. Provide construction notes.
13. Document the results of our conclusions and recommendations in a written geotechnical engineering report.

SITE CONDITIONS

Surface Conditions

The site is located along the southern side of NE 76th Street immediately east of the property located 18459 NE 76th Street. The parcel number for the property is known as 20120906900004. The site is a roughly rectangular shaped parcel covering approximately 3.02 acres. The site is currently undeveloped and vegetated with grass and underbrush. Large soil stockpiles from previous grading activities are located within the southern central portion of the property. Approximately 2 Horizontal to 1 Vertical (2H:1V) slopes are also located on the western and northern sides of the site as shown on Cross Section A-A’ in Figure 3. Based on our experience with the neighboring site to the south, we understand that the soil stockpiles and graded slopes were created during past grading and filling activities within the site. Elevations within the site range from 55 feet within the lower western portion of the site to 109 feet at the top of the soil stockpile within the south-central portion of the property. An approximate elevation contour of 90 feet is located along the toe of the soil stockpile and the top of the graded slopes within the western and northern portions of the property. We did not observe any ponding surface water or groundwater seepage emitting from the site slopes during our site visit on August 31, 2016. We also did not observe signs of recent slope movement on the site fill slopes.

Subsurface Conditions

Geology: The geologic units for this area are shown on the Geologic Map of the Redmond Quadrangle, King County, Washington, by James P. Minard and Derek B. Booth (US Geological Survey, 1988). The site is mapped as Redmond Delta (Qvrd). These deposits are described as sand with gravel soils. In general, our explorations along with the previous exploration performed within the site encountered silty sand with gravel that we interpreted as previously placed structural fill during previous grading activities.

Explorations: The subsurface conditions within the site were explored on August 31, 2016 by excavating eleven test pits to depths ranging from 4.0 to 8.0 feet below the existing surface using a trackhoe. The approximate locations of our explorations are shown on the Schematic Site Plan in Figure 2. A geologist from NGA was present during the explorations, examined the soils and geologic conditions encountered, obtained samples of the different soil types, and maintained logs of the test pits.

The soils were visually classified in general accordance with the Unified Soil Classification System, presented in Figure 4. The test pit logs are attached to this report and are presented as Figures 5 through 8. We present a brief summary of the subsurface conditions in the following paragraph. For a detailed description of the subsurface conditions, the test pit logs should be reviewed.

In all of our test pits, we encountered a surficial layer of grass, roots and gravel surfacing underlain by medium dense to dense, brown to gray silty fine to medium sand with varying amounts of gravel, organics and debris interpreted to be previously placed structural fill soils. All of our test pits were terminated within the previously placed structural fill soils at depths in the range of 4.0 to 8.5 feet below the existing ground surface.

Deeper subsurface boring explorations were performed within the eastern portion of the property by Kleinfelder in 2015. This exploration program consisted of nine drilled borings extending down to depths of 26.5 to 46.5 feet below the existing ground surface. These borings generally encountered undocumented fill soils consisting of lean clay, clayey sand, sandy silt, silty sand, and silty gravel with varying amounts of cobbles, boulders, organics, and wood debris within the upper portion of the borings. Seven of the nine borings were completed within the fill soils. Within the two northern borings, sands and gravels interpreted to be native recessional outwash were encountered at approximately 40 feet below the existing ground surface or an elevation of 45 feet. These two borings were terminated within the native recessional outwash soils.

Hydrogeologic Conditions

Groundwater seepage was not encountered in the explorations. We also did not observe groundwater emitting from the site slopes. The groundwater table on this site is interpreted to be well below the planned wall subgrade elevations. Any groundwater encountered on this site would likely be interpreted as a perched water condition. Perched water occurs when surface water infiltrates through less dense, more permeable soils and accumulates on top of underlying, less permeable soils. Perched water does not represent a regional groundwater "table" within the upper soil horizons. Perched water tends to vary spatially and is dependent upon the amount of precipitation. We would expect the amount of perched water to decrease during drier times of the year and increase during wetter periods.

SENSITIVE AREA EVALUATION

Seismic Hazard

We reviewed the 2012 International Building Code (IBC) for seismic site classification for this project. We anticipate that medium dense to dense glacial soils underlie the site at depth and the site conditions best fit the IBC description for Site Class D.

Hazards associated with seismic activity include liquefaction potential and amplification of ground motion by soft deposits. Liquefaction is caused by a rise in pore pressures in a loose, fine sand deposit beneath the groundwater table. It is our opinion that the competent glacial soils interpreted to underlie the site at depth have a low potential for liquefaction or amplification of ground motion due to their generally medium dense or better condition. Adequate foundation subgrade improvements as discussed in this report should limit the potential impacts of liquefaction induced settlement on the proposed structures due to the existing fill soils within the site.

The criteria used for determination of the erosion hazard for affected areas include soil type, slope gradient, vegetation cover, and groundwater conditions. The erosion sensitivity is related to vegetative cover and the specific surface soil types, which are related to the underlying geologic soil units. The Soil Survey of King County Area, Washington, by the Soil Conservation Service (SCS), was reviewed to determine the erosion hazard of the on-site soils. The surface soils for this site are mapped as Everett very gravelly sandy loam, 8 to 15 percent slopes. The erosion hazards for these soil types is listed as slight and moderate. We anticipate that the existing fill soils within the site have a moderate to severe erosion hazard.

Slope Stability Analysis

The proposed Hilfiker retaining walls were analyzed for stability along three representative sections of the proposed wall alignment, using the computer program Slope/W, by Geo-Slope International. Slope/W is a two-dimensional, limit equilibrium slope stability program that generates random potential failure surfaces or specific failure surfaces and determines their corresponding factors of safety with respect to failure. By generating a large number of random surfaces, a critical failure surface with the minimum factor of safety can be identified.

The slope stability analyses were performed using information gathered from the field explorations and review of the previous reports prepared for the site, and soil properties were assigned to the soil layers to reasonably reflect their engineering characteristics. Stability analyses were performed at the three cross-sections for non-seismic and seismic conditions for the proposed conditions. These three cross sections included the proposed single tier wall along the northern portion of the site and two from the two-tiered section of the wall along the western portion of the property (one below the proposed concrete retaining wall and building and one below the proposed parking area). A peak ground acceleration of 0.15g was used in the seismic analyses. The soil parameters used in our analyses, along with the results of the analyses, are presented in Figures 9 through 14.

Our global slope stability analyses of the proposed Hilfiker Retaining Walls as discussed in this report achieved appropriate minimum factors of safety of 1.5 for static conditions and 1.2 for seismic loading.

CONCLUSIONS AND RECOMMENDATIONS

General

It is our opinion that the site is compatible with the planned improvements from a geotechnical standpoint. Our explorations indicated that the site is underlain by as much as 50 feet of previously placed fill soils. This fill is generally in a medium dense to dense condition. We understand that the proposed building will likely be supported on both newly placed structural fill associated with the retaining wall construction on the western portion of the property and the existing undocumented fill soils within the eastern portion of the property. To minimize the potential for foundation settlement and other settlement-related problems, we recommend that that the portion of the building foundations located within the existing undocumented fill soils within the eastern portion of the site be overexcavated by a minimum of four feet and the overexcavation be backfilled with two, two-foot thick layers of crushed rock fully wrapped with a Tensar TX160 geogrid up to the foundation subgrade. The eastern portion of the building outside of the newly placed structural fill associated with the retaining walls within the western portion of the property should be directly supported on the crushed rock wraps. Foundations within the newly placed structural fill along the western portion of the property could be directly on the newly placed structural fill.

Alternatively, the structure foundation loads could be supported on modified ground in the form of compacted rock aggregate piers. This type of ground modification includes augered or vibratory excavations within the foundation and slab areas extending through the upper loose/soft soils. Compacted crushed rock fill is placed within the excavations to further densify the upper loose/soft soils and reducing the potential for future settlement within the building elements. The building foundations and slabs are then supported directly on the compacted rock aggregate piers

If settlement of the slab on grade and future slab maintenance can be tolerated, the undocumented fill material found within the slab area should be over-excavated approximately two feet and replaced with crushed rock. If settlement of the slab on grade and future slab maintenance cannot be tolerated, the slab should be supported on the geogrid wraps or the compacted rock aggregate piers. If the slab is supported on aggregate piers, the slab should be designed as a structural element, and sized and reinforced accordingly. If the slab is supported on a crushed rock layer, added rebar and cold joints should be incorporated into the slab design to reinforce the slab and minimize damage caused by potential future settlement.

It is also our opinion reinforced-earth retaining walls could be constructed along the toe of the site slope along the western and northern portions of the property to bring the site up to the proposed finished grade elevations. In our opinion, a Hilfiker Retaining Wall system is compatible with site conditions along the western and northern portions of the site and a geo-grid reinforced Keystone block retaining wall is suitable for the site conditions along northeastern portion of the building supporting the upper parking lot area. The Hilfiker Retaining Wall system utilizes welded wire matting as the wall facing and for reinforcing the wall backfill.

Our explorations and review of the previous explorations within the site generally indicated that the planned wall area and site slopes are generally underlain by previously placed competent structural fill soils. The medium dense or better fill soils should provide adequate support for the planned retaining walls. The foundations for the retaining walls should extend through any undocumented fill, organic soil, or loose materials, and be keyed into the underlying medium dense or better native soils or competent structural fill. The subgrade for the proposed Hilfiker walls should then be overexcavated by a minimum of two feet and backfilled with 2- to 4-inch rock spalls. We recommend that level benches be graded into the site slope to allow for placement of the wall components and fills to be retained by the walls. We recommend that the individual wall tiers have a maximum exposed height of 10.0 feet. We anticipate that the total individual tier height may be up to 12.0 feet in order to satisfy the recommended base embedment into the native soils. This is discussed further in the **Hilfiker Retaining Wall Design and Construction Recommendations** subsection of this letter. NGA should be retained to review project plans prior to construction and should be retained to observe wall construction to verify wall installation is performed in accordance with the plans and our recommendations.

Subgrade preparation in the pavement areas should consist of over-excavating by a minimum of one foot and replaced with crushed rock. The crushed rock should be compacted to structural fill specifications prior to placing pavement. We recommend that the exposed subgrade be compacted to a non-yielding condition using a heavy vibratory drum roller prior to placing crushed rock. The resulting surface should be proof-rolled using a loaded dump truck. Areas observed to pump or weave during the proof-roll test should be over-excavated and replaced with structural fill. Depending on the material exposed in the excavation, a layer of geogrid may need to be placed over the exposed surface prior to placing structural fill. This can be determined in the field based on actual conditions. Once a stable subgrade is achieved, the structural fill could be placed over the prepared subgrade.

The soils that are expected to be encountered during site development are considered highly moisture-sensitive and will disturb in wet conditions. We recommend that the site be developed during the dry season. If construction takes place during the rainy months, the site soils may disturb and become extremely difficult to work. Also, if construction takes place during the wet season, additional expenses and delays should be expected. Additional expenses could include the need for placing a blanket of rock spalls on exposed subgrades, construction traffic areas, and pavement areas prior to placing structural fill. NGA should be retained to determine if some of the on-site soils could be used as structural fill material during construction.

All grading operations and drainage improvements planned as part of this project should be planned and completed in a manner that enhances the stability of the slope, not reduces it. Any excavation spoils generated during site improvements should not be stockpiled on site but rather promptly hauled away. Also, all current and future runoff generated within the site should be collected and routed to a permanent discharge location at the bottom of the slope, or to an approved drainage system. Under no circumstances should water be allowed to concentrate or flow uncontrollably over the walls or slope. The vegetation cover on the slope should be evaluated for compatibility with desired slope stability conditions, and a vegetation management plan should be devised to enhance slope stability.

Erosion Control and Slope Protection

The on-site soils have a moderate to high potential for water erosion when cleared of vegetation, but the actual erosion potential will be dependent on how the site is graded and how water is allowed to concentrate. Best Management Practices (BMPs) should be used to control erosion. Areas disturbed during construction should be protected from erosion. Erosion control measures may include diverting surface water away from the stripped areas. Silt fences or straw bales could be erected to prevent muddy water from flowing off the site. Stockpiles should be covered with plastic sheeting. Disturbed areas should be planted as soon as practical and the vegetation should be maintained until it is established. The erosion potential for areas not stripped of vegetation should be low to moderate. Disturbed areas outside of the proposed development areas should be replanted with vegetation at the end of construction. The vegetation should be maintained until it is established. Final grading should incorporate permanent erosion control measures and should be designed to route stormwater runoff to appropriate discharge locations away from the structures and sloping ground.

Temporary and Permanent Slopes

Cuts associated with over-excavation of foundation and slab areas may be used for this project. Temporary cut slope stability is a function of many factors, including the type and consistency of soils, depth of the cut, surcharge loads adjacent to the excavation, length of time a cut remains open and the presence of surface or groundwater. It is exceedingly difficult under these variable conditions to estimate a stable, temporary, cut slope angle. Therefore, it should be the responsibility of the contractor to maintain safe slope configurations since they are continuously at the job site, able to observe the nature and condition of the cut slopes, and able to monitor the subsurface materials and groundwater conditions encountered.

The following information is provided solely for the benefit of the owner and other design consultants and should not be construed to imply that Nelson Geotechnical Associates, Inc. assumes responsibility for job site safety. Job site safety is the sole responsibility of the project contractor.

For planning purposes, we recommend that temporary cuts in the on-site soils be no steeper than 2 Horizontal to 1 Vertical (2H:1V) if worker access is necessary. If significant groundwater seepage is encountered, we would expect that flatter inclinations would be necessary. We recommend that cut slopes be protected from erosion. These erosion protection measures may include covering cut slopes with plastic sheeting and diverting surface runoff away from the top of cut slopes. We do not recommend vertical slopes for cuts deeper than four feet, if worker access is necessary. We recommend that cut slope heights and inclinations conform to appropriate OSHA/WISHA regulations.

Site Preparation and Grading

After erosion control measures are implemented, site preparation should consist of stripping the proposed building, retaining wall and pavement areas of all organics, loose undocumented fill, to expose medium dense or better soils. The resulting foundation subgrade areas should then be overexcavated as discussed in this report and replaced with geogrid-reinforced structural fill. The resulting subgrade should be proof-rolled and repaired to achieve a non-yielding state. Level benches should be created for the retaining wall and associated backfill placement.

If the ground surface, after site stripping, should appear to be loose, it should be compacted to a non-yielding condition and then proof-rolled with a heavy rubber-tired piece of equipment. Areas observed to pump or weave during the proof-roll test should be reworked to structural fill specifications or over-excavated and replaced with properly compacted structural fill or rock spalls. If soft soils are encountered in the building or pavement areas, these areas should be removed and replaced with rock spalls or granular structural fill as discussed in this report. If significant surface water flow is encountered during construction, this flow should be diverted around areas to be developed, and the exposed subgrades should be maintained in a semi-dry condition.

If wet conditions are encountered, alternative site stripping and grading techniques might be necessary. These could include using large excavators equipped with wide tracks and a smooth bucket to complete site grading and covering exposed subgrade with a layer of crushed rock for protection. If wet conditions are encountered or construction is attempted in wet weather, the subgrade should not be compacted as this could cause further subgrade disturbance. In wet conditions it may be necessary to cover the exposed subgrade with a layer of crushed rock as soon as it is exposed to protect the moisture sensitive soils from disturbance by machine or foot traffic during construction. The prepared subgrade should be protected from construction traffic and surface water should be diverted around prepared subgrade.

The site soils are considered to be moisture-sensitive and can disturb easily when wet. We recommend that construction take place during the drier summer months if possible. However, if construction takes place during the wet season, additional expenses and delays should be expected due to the wet conditions.

Additional expenses could include the need for placing a blanket of rock spalls on exposed subgrades, construction traffic areas, and paved areas prior to placing structural fill. The successful use of on-site soils as structural fill will be very difficult, but will depend on the moisture content of the soil at the time of construction. NGA should be retained to determine if any of the on-site soils could be used as structural fill material prior to construction.

Foundations

Shallow Spread Footings: Conventional shallow spread foundations should be placed on either newly placed structural fill associated with bringing up the site grades within the western portion of the property or on geo-grid wrapped crushed rock layers within the eastern portion of the building within the existing fill soils. In the areas of the building to be founded within the existing fill soils, we recommend that the building and retaining wall foundation areas be over-excavated by a minimum of four feet below the bottom of planned foundations. The over-excavation should also extend a minimum of four feet beyond the perimeter of the proposed foundations. If pockets of organic rich soils are encountered at the base of the over-excavation, we recommend that these soils be removed and replaced with crushed rock. The bottom of the over-excavation should then be covered with a layer of Tensar TX 160 geogrid and 24-inches of crushed rock fill that is fully wrapped with the geogrid. A second 24-inch crushed rock layer wrapped with geogrid should be placed on top of the lower wrap up to the foundation sugrade. Geogrid sections should overlap a minimum of 18-inches. If the foundation excavation needs to be stepped down in any location and geogrid wraps will be overlapped, we recommend that six inches of pit run be placed in between the wraps, and the stepped down wraps be overlapped by a minimum of three feet. We have provided a schematic detail of the recommended foundation subgrade preparation in Figure 21. We should be retained to observe foundation subgrade preparation and provide supplemental recommendations as needed.

Conventional shallow spread foundations could also be supported on compacted rock aggregate piers as opposed to over-excavation of the fill soils. Rock aggregate piers consist of augered or vibratory excavations holes which are then backfilled in successive lifts with crushed gravel which is rammed into place to create a dense column in which footings can bear on. The manner at which the material is packed into the hole also densifies the surrounding soils. We recommend that the piers consist of a minimum 16-inch diameter rock aggregate piers placed at 5 feet on center along the footing lines to provide proper support for the footings in the unsuitable soils. Isolated foundations should also be supported directly on a grid of compacted rock aggregate piers. Based on explorations performed within the site, the piers would need to extend up to 50 feet below the existing ground surface to reach the lower native glacial soils below. The rock aggregate pier layout and overall design are generally performed by a rock aggregate pier company and their structural engineer.

Footings should extend at least 18 inches below the lowest adjacent finished ground surface for frost protection and bearing capacity considerations. Foundations should be designed in accordance with the 2015 IBC. Minimum foundation widths of 18 and 24 inches should be used for continuous and isolated footings, respectively. Footing widths should also be based on the anticipated loads and allowable soil bearing pressure. Standing water should not be allowed to accumulate in footing trenches. All loose or disturbed soil should be removed from the foundation excavation prior to placing concrete.

For foundations constructed as outlined above, we recommend an allowable design bearing pressure of not more than 2,000 pounds per square foot (psf) be used for the footing design for footings founded on the medium dense or better native soils at depth, compacted rock aggregate piers or CDF extending to the native competent material. The foundation bearing soil should be evaluated by a representative of NGA. We should be consulted if higher bearing pressures are needed. Current IBC guidelines should be used when considering increased allowable bearing pressure for short-term transitory wind or seismic loads. Potential foundation settlement using the recommended allowable bearing pressure is estimated to be less than one inch total and 1/2 inch differential between adjacent footings or across a distance of about 30 feet.

Lateral loads may be resisted by friction on the base of the footing and passive resistance against the subsurface portions of the foundation. A coefficient of friction of 0.35 may be used to calculate the base friction and should be applied to the vertical dead load only. Passive resistance may be calculated as a triangular equivalent fluid pressure distribution. An equivalent fluid density of 150 pounds per cubic foot (pcf) should be used for passive resistance design for a level ground surface adjacent to the footing. This level surface should extend a distance equal to at least three times the footing depth. These recommended values incorporate safety factors of 1.5 and 2.0 applied to the estimated ultimate values for frictional and passive resistance, respectively. To achieve this value of passive resistance, the foundations should be poured "neat" against the existing soils or compacted fill should be used as backfill against the front of the footing. We recommend that the upper one-foot of soil be neglected when calculating the passive resistance.

Hilfiker Retaining Wall Design and Construction Recommendations

The Hilfiker Wall system utilizes welded wire mats that make up the wall face and reinforce the soil by extending back into the wall backfill. The tiered wall designs within the western portion of the property includes individual wall heights ranging from 4 to 12 feet, with reinforced mat lengths of up to 21 feet to maintain global stability. The single tier wall design within the northern portion of the property includeds an individual wall height of up to 26 feet, with reinforced mat lengths of up to 24 feet to maintain global stability. The provided Hilfiker wall design should be reviewed in detail for a complete understanding of our recommendations. The wall profile and locations of the corresponding matting placement, lengths, and matting weights should be all determined by a qualified civil engineer or contractor prior to ordering wall material. A detailed description of the wall design and installation recommendations is presented as Figures 15 through 19. All recommendations presented on these figures regarding mat placement and fill material and compaction methods should be strictly followed. We should be onsite during wall construction.

The proposed area where the wall and reinforced fill will be placed should be cleared of any loose material and be made level. Additional benching of the slope above the wall area will be needed to maintain safe temporary cut excavations to facilitate installation of the wall and backfill and to key the backfill into the existing slope. The benches will need to be level and have a minimum width of six feet. This is extremely important to maintain long-term stability of the wall and fill mass.

We should note that ideally free draining backfill would be placed within the reinforced fill zone, however, granular material with up to 20 percent silt contents could be used as approved by NGA. Depending on the material utilized and actual conditions in the field, surface and/or subsurface drains may be required. This can be better evaluated in the field at the time of construction. The wall backfill and any other fills placed on this site should be placed and compacted as described in the **Structural Fill** subsection of this report.

A crushed rock layer is recommended along the face of the wall directly behind the hardware cloth for the purpose of facilitating drainage and also reducing the potential for bulging of the wall face. Face bulging may still occur; however this condition is normal and should not lead to any structural damage to the wall. It is also our understanding that a “green” wall may be desired. That should be adequate.

The proposed wall subgrade should be prepared by removing all surficial unsuitable soil, undocumented fill and organic rich soils, exposing competent soils. The wall subgrade should be over-excavated by a minimum of two feet and replaced with 2- to 4-inch rock spalls. We understand that the base of the lower retaining walls will be raised. We recommend that the wall be embedded a minimum of 18-inches into rock spalls, as shown on the Cross-Section in Figure 15. We should be retained to observe and verify conditions encountered during wall construction and evaluate the exposed wall subgrade.

Keystone Block Retaining Wall

The total height of the new block wall will vary somewhat, but we understand that it will generally be up to approximately twelve feet tall, including the recommended minimum embedment of one foot into competent soils. We have provided a wall design for an up to 12-foot-tall total retaining wall with geogrid-reinforced fill utilizing CornerStone R-100 blocks or 21.5-inch Standard Keystone blocks. A geogrid-reinforced wall detail and construction notes are shown in Figure 20. The retained fill zone should consist of imported granular material compacted to structural fill specifications. The drainage system, as indicated on the detail, should be installed along the base of the blocks and behind the wall facing.

The block facing should consist of CornerStone R-100 or 21.5-inch Standard Keystone blocks. The block facing should be placed on a minimum of 6-inch thick crushed rock leveling pad placed over competent native soils. The subgrade should be level and compacted to a non-yielding condition before placing the blocks or backfill. If loose undocumented fill soils are encountered at the foundation subgrade elevation of the retaining wall, we recommend that the foundation area be over-excavated down to medium dense or better native soils.

A drainage blanket of 12 inches of free-draining ¾-inch clean crushed rock should be placed between the blocks and the retained fill zone. The block cavities should also be filled with the crushed rock. A rigid, 4-inch perforated drainpipe embedded in a minimum of one foot of drain rock and wrapped in a filter fabric should be placed at the bottom of the drainage blanket. The drain should be sloped to drain into an existing system below the retaining wall or daylighted through the wall a minimum of every 50 feet.

Mirafi 5XT geogrid (or equivalent) is recommended to be used in the geogrid-reinforced fill wall design. The geogrid should be cut to the recommended lengths, attached to the blocks as recommended by the manufacturer, and extended back into the reinforced fill zone. The grid should be pulled tight before the fill is placed over the geogrid. Care should be taken to not damage the geogrid by operating construction equipment on the exposed grid, or by allowing large rocks to be placed directly on the grid.

All fill placed in the retained fill zone behind the retaining walls should be placed as structural fill. Structural fill, by definition, is placed in accordance with prescribed methods and standards and is monitored by an experienced geotechnical professional or soils technician. Field monitoring procedures would include the performance of a representative number of in-place density tests to document the attainment of the desired degree of relative compaction. The fill subgrade should consist of native medium dense or better native soil compacted to a non-yielding condition. The area to receive block or wall backfill should be excavated into a level bench and be free of all fill soils or loose material.

Structural fill should consist of a good quality, imported granular soil, free of organics and other deleterious material and be well graded to a maximum size of about three inches. The material should have no more than 20 percent by weight of the portion passing the US #200 Sieve. The on-site soils are likely unsuitable for use as wall backfill due to the presence of large cobbles and boulders, however can be used to backfill behind the reinforced soil zone. We should be retained to evaluate proposed fill material prior to construction.

Following subgrade preparation, placement of structural fill may proceed. All fill placements should be accomplished in uniform lifts up to eight inches thick. Each lift should be spread evenly and be thoroughly compacted prior to placement of subsequent lifts. All structural fill should be compacted to a minimum of 95 percent of the material’s maximum dry density. Maximum dry density, in this report, refers to that density as determined by the ASTM D 1557 Compaction Test procedure. The moisture content of the soils to be compacted should be within about two percent of optimum so that a readily compactable condition exists. It may be necessary to over-excavate and remove wet soils in cases where drying to a compactable condition is not feasible. All compaction should be accomplished by equipment of a type and size sufficient to attain the desired degree of compaction.

If groundwater seepage is encountered or if excessive rainfall occurs during construction of specific aspects, we recommend that the contractor slope the bottom of the excavations and direct the water to ditches and small sump pits. The collected water can then be directed to a suitable discharge point.

Other Retaining Walls

The lateral pressure acting on subsurface retaining walls is dependent on the nature and density of the soil behind the wall, the amount of lateral wall movement which can occur as backfill is placed, wall drainage conditions, and the inclination of the backfill. For walls that are free to yield at the top at least one thousandth of the height of the wall (active condition), soil pressures will be less than if movement is limited by such factors as wall stiffness or bracing (at-rest condition). We recommend that walls supporting horizontal backfill and not subjected to hydrostatic forces, be designed using a triangular earth pressure distribution equivalent to that exerted by a fluid with a density of 45 pcf for yielding (active condition) walls, and 65 pcf for non-yielding (at-rest condition) walls.

These recommended lateral earth pressures are for a drained granular backfill and are based on the assumption of a horizontal ground surface behind the wall for a distance of at least the subsurface height of the wall, and do not account for surcharge loads. Additional lateral earth pressures should be considered for surcharge loads acting adjacent to subsurface walls and within a distance equal to the subsurface height of the wall. This would include the effects of surcharges such as traffic loads, floor slab loads, slopes, or other surface loads. We could consult with the structural engineer regarding additional loads on retaining walls during final design, if needed.

The lateral pressures on walls may be resisted by friction between the foundation and subgrade soil, and by passive resistance acting on the below-grade portion of the foundation. Recommendations for frictional and passive resistance to lateral loads are presented in the **Foundations** subsection of this report.

All wall backfill should be well compacted as outlined in the **Structural Fill** subsection of this report. Care should be taken to prevent the buildup of excess lateral soil pressures due to over-compaction of the wall backfill. This can be accomplished by placing wall backfill in 8-inch loose lifts and compacting the backfill with small, hand-operated compactors within a distance behind the wall equal to at least one-half the height of the wall. The thickness of the loose lifts should be reduced to accommodate the lower compactive energy of the hand-operated equipment. The recommended level of compaction should still be maintained.

Permanent drainage systems should be installed for retaining walls. Recommendations for these systems are found in the **Subsurface Drainage** subsection of this report. We recommend that we be retained to evaluate the proposed wall drain backfill material and observe installation of the drainage systems.

Structural Fill

General: Fill placed behind the wall and within the slope area should be placed as structural fill. Structural fill, by definition, is placed in accordance with prescribed methods and standards, and is monitored by an experienced geotechnical professional or soils technician. Field monitoring procedures would include the performance of a representative number of in-place density tests to document the attainment of the desired degree of relative compaction. The area to receive the fill should be suitably prepared as described in the **Site Preparation and Grading** subsection of this report, prior to beginning fill placement. Sloping areas on this site should be benched for fill placement. The benches should be level and be a minimum of six feet in width.

Materials: Structural fill should consist of a good quality, granular soil, free of organics and other deleterious material and be well graded to a maximum size of about three inches. In wet weather, the fill material should contain no more than five-percent fines (soil finer than U.S. No. 200 sieve, based on that fraction passing the U.S. 3/4-inch sieve). The use of some on-site soils as structural fill should be feasible provided the proposed fill material contains no more than 20 percent silt and can be moisture-conditioned to near optimum moisture content for compaction. All organic matter and debris should be removed form on-site soils planned for use as structural fill. NGA should be retained during construction to determine if any of the on-site soils could be used as structural fill.

Fill Placement: Following subgrade preparation, placement of structural fill may proceed. All backfilling should be accomplished in uniform lifts up to eight inches thick. Each lift should be spread evenly and be thoroughly compacted prior to placement of subsequent lifts. All structural fill should be compacted to a minimum of 95 percent of its maximum dry density. Maximum dry density, in this report, refers to that density as determined by the ASTM D-1557 Compaction Test procedure. The moisture content of the soils to be compacted should be within about two percent of optimum so that a readily compactable condition exists. It may be necessary to over-excavate and remove wet soils in cases where drying to a compactable condition is not feasible. All compaction should be accomplished by equipment of a type and size sufficient to attain the desired degree of compaction.

Slab-on-Grade

Slabs-on-grade could be “floated” on the undocumented fill soils within the site, if some risk of settlement and maintenance of the slabs is acceptable. If the settlement risk is acceptable, the slab subgrade should be prepared by over-excavating the subgrade by a minimum of two feet below finished grade and backfilling the excavation with 2-inch clean crushed rock. Depending on the material exposed in the over-excavation, it might be necessary to place a layer of geogrid on the exposed surface prior to placing the crushed rock. This can be determined in the field based on site conditions. The slabs should be additionally reinforced, and cold joints incorporated in the slab design to further reduce the effects of differential settlement. If no slab settlements can be tolerated, the slab should be designed as a structural element and supported on aggregate piers. The aggregate piers within the slab should be spaced no more than 8 feet on center.

We recommend that all floor slabs be underlain by at least six inches of free-draining sand or gravel for use as a capillary break. We recommend that the capillary break be hydraulically connected to the footing drain system to allow free drainage from under the slab. A suitable vapor barrier, such as heavy plastic sheeting (6-mil minimum), should be placed over the capillary break material.

Pavements

Pavement subgrade preparation, and structural fill placement where required, should be completed as recommended in the **Site Preparation and Grading** and **Structural Fill** subsections of this report. We recommend that a minimum of one foot of crushed rock be placed below the pavement section. The existing soil should be over excavated and replaced with crushed rock fill prior to placing new pavement section. The pavement subgrade should be heavily compacted and proof-rolled with a heavy, rubber-tired piece of equipment, to identify soft or yielding areas that require repair prior to placing structural fill and prior to placing the pavement base course. A layer of geofabric may need to be placed over the exposed subgrade prior to placing the structural fill. We should be retained to observe the proof-rolling and recommend subgrade repairs prior to placement of the asphalt or hard surfaces.

Utilities

We recommend that underground utilities be bedded with a minimum 12 inches of pea gravel prior to backfilling the trench with on-site or imported material meeting structural fill requirements. Trenches within settlement sensitive areas should be compacted to 95% of the modified proctor as described in the **Structural Fill** subsection of this report. Trenches located in non-structural areas should be compacted to a minimum 90% of the maximum dry density. When excessively soft and/or debris-laden soils are encountered within utility trench excavations, such soils should be overexcavated by a minimum of 12 inches and replaced with crushed rock.

Site Drainage

Surface Drainage: The finished ground surface should be graded such that stormwater is directed to an appropriate stormwater collection system. Water should not be allowed to stand in any area where footings or slabs are to be constructed. Final site grades should allow for drainage away from the structures. We suggest that the finished ground be sloped at a minimum gradient of three percent, for a distance of at least 10 feet away from the structures. Surface water should be collected by permanent catch basins and drain lines, and be discharged into an appropriate discharge system.

Subsurface Drainage: If groundwater is encountered during construction, we recommend that the contractor slope the bottom of the excavation and collect the water into ditches and small sump pits where the water can be pumped out of the excavation and routed into an appropriate discharge point.

We recommend the use of footing drains around the planned structure. Footing drains should be installed at least one foot below planned finished floor elevation. The drains should consist of a minimum four-inch-diameter, rigid, slotted or perforated, PVC pipe surrounded by free-draining material, such as washed rock, wrapped in a filter fabric. We recommend that an 18-inch-wide zone of clean (less than three-percent fines), granular material be placed along the back of subsurface walls above the drain. Pea gravel is an acceptable drain material, or drainage composite may be used instead. The free-draining material should extend up the wall to one foot below the finished surface. The top foot of backfill should consist of impermeable soil placed over plastic sheeting or building paper to minimize the migration of surface water or fines into the footing drain. Footing drains should discharge into tightlines leading to an appropriate collection and discharge point with convenient cleanouts to prolong the useful life of the drains. Roof drains should not be connected to footing drains.

USE OF THIS REPORT

NGA has prepared this report for Mr. J.J. Engler and his agents, for use in the planning and design of the development planned on this site only. The scope of our work does not include services related to construction safety precautions and our recommendations are not intended to direct the contractors' methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design. There are possible variations in subsurface conditions between the explorations and also with time. Our report, conclusions, and interpretations should not be construed as a warranty of subsurface conditions. A contingency for unanticipated conditions should be included in the budget and schedule.

We recommend that NGA be retained to review project plans and consult with the design team during final design. We also recommend that NGA be retained to provide monitoring and consultation services during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork and foundation installation activities comply with contract plans and specifications. We should be contacted a minimum of one week prior to construction activities and could attend pre-construction meetings if requested.

Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering practices in effect in this area at the time this report was prepared. No other warranty, expressed or implied, is made. Our observations, findings, and opinions are a means to identify and reduce the inherent risks to the owner.

o-o-o

It has been a pleasure to provide service to you on this project. If you have any questions or require further information, please call.

Sincerely,

NELSON GEOTECHNICAL ASSOCIATES, INC.

Lee S. Bellah, LG
Project Geologist

Khaled M. Shawish, PE
Principal

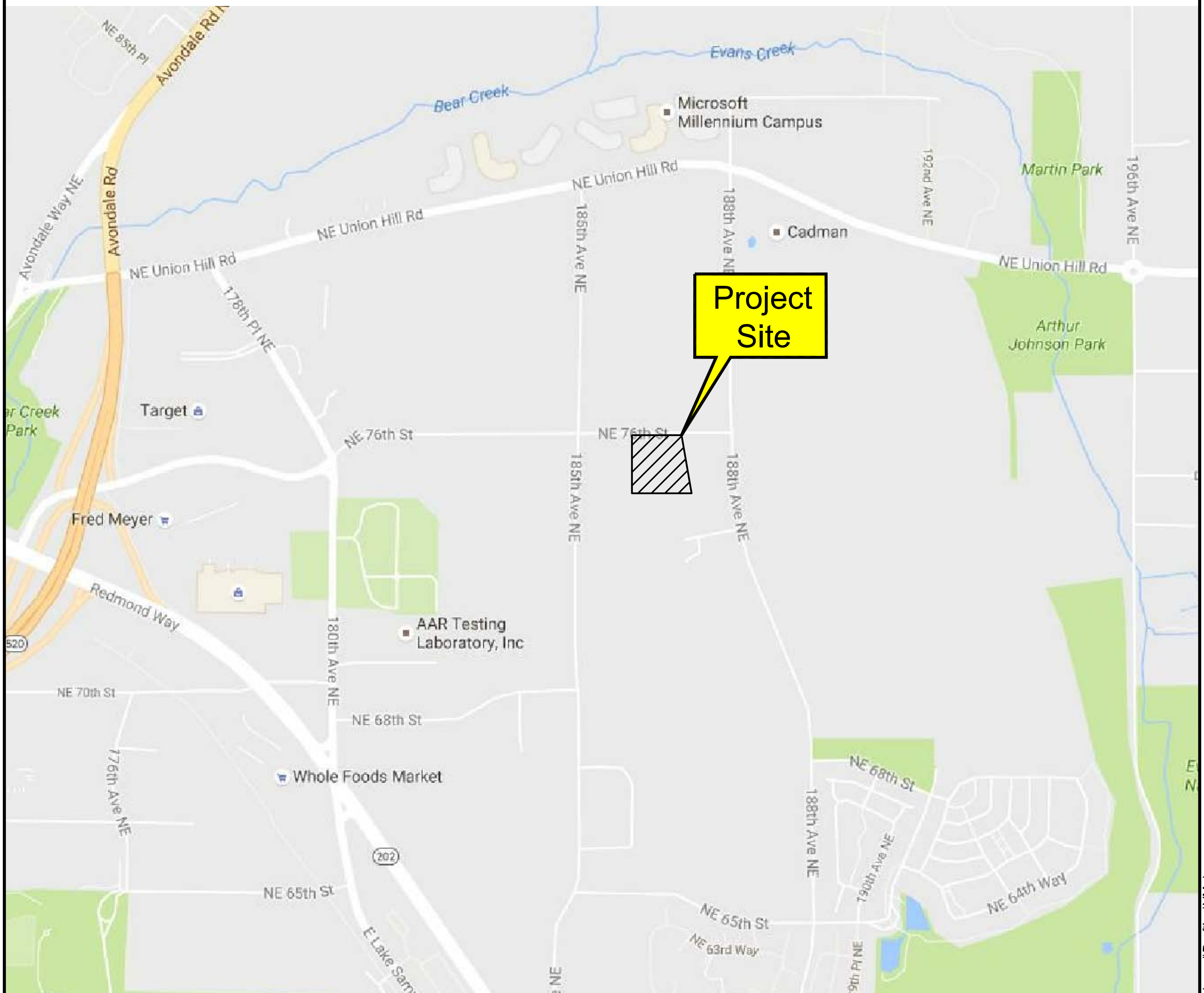
LSB:KMS:dy

Twenty-One Figures Attached

VICINITY MAP

Not to Scale

N



Redmond, WA

Project Number 969616	Union Hill Self-Storage Vicinity Map	NELSON GEOTECHNICAL ASSOCIATES, INC. GEOTECHNICAL ENGINEERS & GEOLOGISTS	No.	Date	Revision	By	CK
Figure 1		17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax 481-2510; Snohomish County (425) 339-1669 Wenatchee/Chelan (509) 665-7696 www.nelsongeotech.com	1	9/7/16	Original	DPN	ABR

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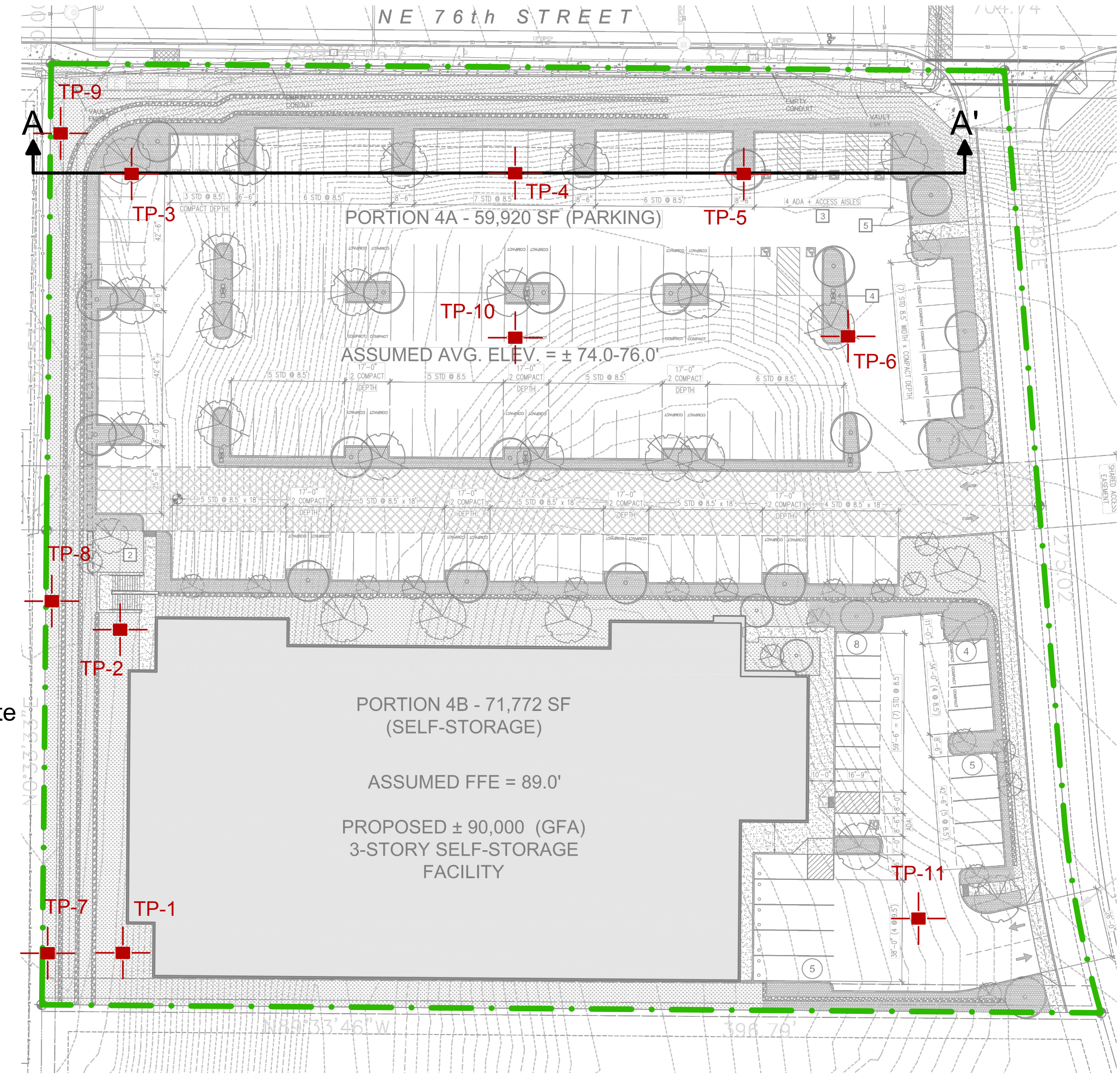
Site Plan



N

LEGEND

Property line

TP-1

Number and approximate location of test pit

A A'

Approximate location of cross-section

0 60 120

Scale: 1 inch = 60 feet

Reference: Site plan based on a plan dated July 7, 2016 titled "Union Hill Self-Storage and Auxiliary Parking Lot," prepared by Magellan Architects.

Project Number
969616

Figure 2

Union Hill Storage
Site Plan

NGA
NELSON GEOTECHNICAL
ASSOCIATES, INC.
GEOTECHNICAL ENGINEERS & GEOLOGISTS
17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax 481-2510
Snohomish County (425) 337-1669
Wenatchee/Chelan (509) 665-7696
www.nelsongeotech.com

No.	Date	Revision	By	CK
1	9/7/16	Original	DPN	ABR

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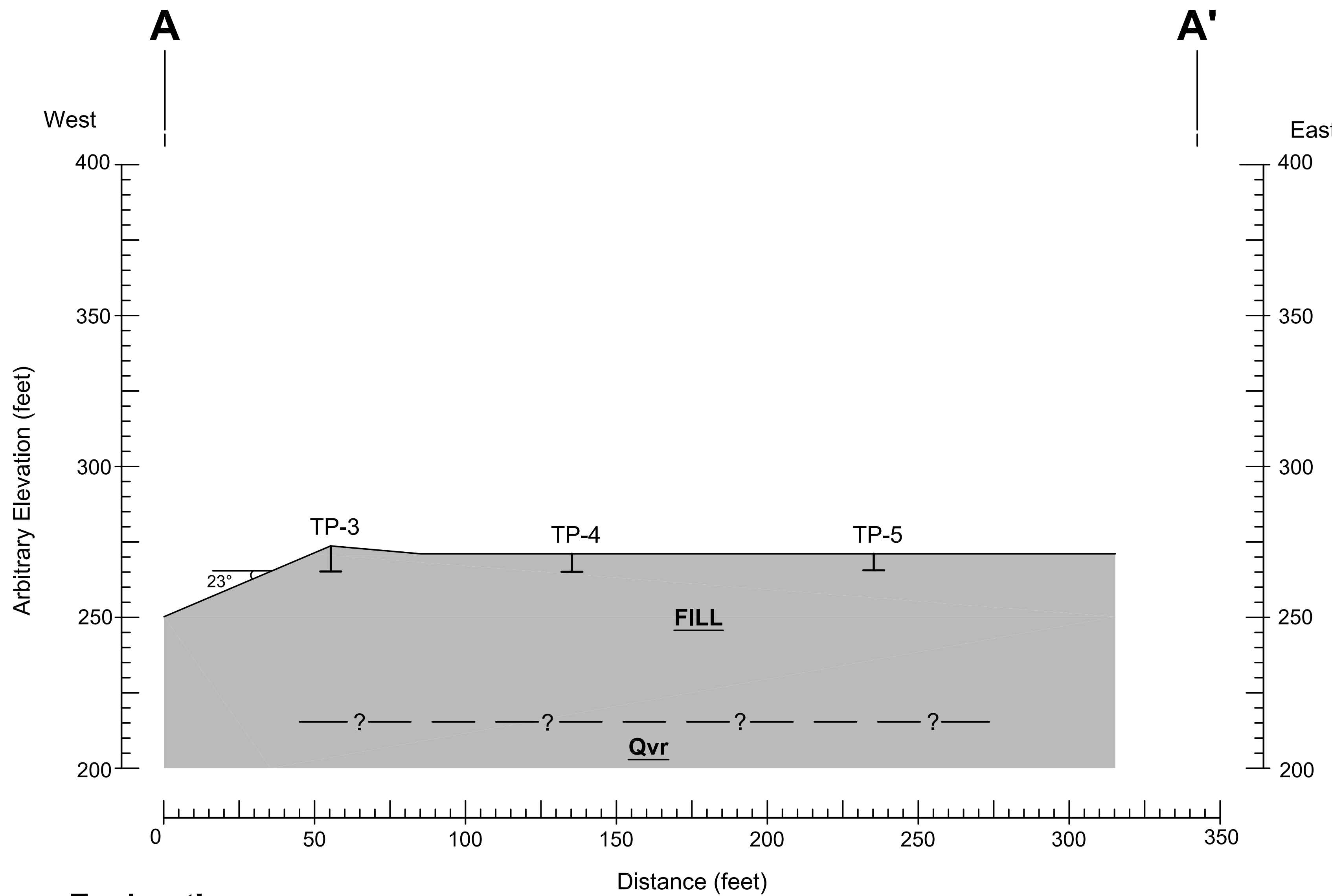

Exploration

Test Pit Designation → TP-1

Groundwater Level During Exploration →

Geologic Contact (approximate) → ? — — ?

NOTES:
1) Stratigraphic conditions are interpolated between the explorations. Actual conditions may vary.
2) Elevations are arbitrary.

Reference: Cross Section is based on field measurements using a hand-held clinometer and 100-ft tape measure.

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Project Number 969616

Figure 3

Union Hill Self-Storage
Cross-Section A-A'

NGA
NELSON GEOTECHNICAL ASSOCIATES, INC.
GEOTECHNICAL ENGINEERS & GEOLOGISTS
17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax 481-2510
437 East Penny Road
Wenatchee, WA 98801
(509) 665-7696

No.	Date	Revision	By	CK
1	9/7/16	Original	DPN	ABR

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE - GRAINED SOILS MORE THAN 50 % RETAINED ON NO. 200 SIEVE	GRAVEL MORE THAN 50 % OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVEL	GW	WELL-GRADED, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND MORE THAN 50 % OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE - GRAINED SOILS MORE THAN 50 % PASSES NO. 200 SIEVE	SILT AND CLAY LIQUID LIMIT LESS THAN 50 %	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY LIQUID LIMIT 50 % OR MORE	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FLAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

NOTES:

1) Field classification is based on visual examination of soil in general accordance with ASTM D 2488-93.

2) Soil classification using laboratory tests is based on ASTM D 2488-93.

3) Descriptions of soil density or consistency are based on interpretation of blowcount data, visual appearance of soils, and/or test data.

SOIL MOISTURE MODIFIERS:

Dry - Absence of moisture, dusty, dry to the touch

Moist - Damp, but no visible water.

Wet - Visible free water or saturated, usually soil is obtained from below water table

Project Number 969616	Union Hill Self-Storage Soil Classification Chart	NELSON GEOTECHNICAL ASSOCIATES, INC. GEOTECHNICAL ENGINEERS & GEOLOGISTS	No.	Date	Revision	By	CK
Figure 4			1	9/7/16	Original	DPN	ABR

NGA
17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax 481-2510
Snohomish County (425) 337-1669
Wenatchee/Chelan (509) 665-7696
www.nelsongeotech.com

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Test pit logs indicate samples were collected at each test pit location. Please provide the sample results.

LOG OF EXPLORATION

DEPTH (FEET)	USC	SOIL DESCRIPTION
TEST PIT ONE		
0.0 – 0.3		GRASS AND ROOTS
0.3 – 4.0		LIGHT BROWN TO BROWN, SILTY FINE TO COARSE SAND WITH GRAVEL, AND WOOD DEBRIS (MEDIUM DENSE TO DENSE, MOIST) (**FILL**)
4.0 – 8.0		GRAY, SILTY FINE TO COARSE SAND WITH GRAVEL, COBBLES, METAL SCRAPS, PLASTIC DEBRIS, AND TRACE CONCRETE CHUNKS (MEDIUM DENSE TO DENSE, MOIST) (**FILL**)
		SAMPLES WERE COLLECTED AT 4.0 AND 8.0 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 8.0 FEET ON 8/31/16
TEST PIT TWO		
0.0 – 0.3		GRASS AND ROOTS
0.3 – 1.3		LIGHT BROWN, SILTY FINE TO MEDIUM WITH ROOTS AND GRAVEL (MEDIUM DENSE TO DENSE, MOIST) (**FILL**)
1.3– 7.5		BROWN TO GRAY, SILTY FINE TO COARSE SAND WITH GRAVEL, COBBLES, WOOD DEBRIS, METAL SCRAPS, AND GARBAGE (MEDIUM DENSE TO DENSE, MOIST) (**FILL**)
		SAMPLES WERE COLLECTED AT 6.0 AND 7.5 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 7.5 FEET ON 8/31/16
TEST PIT THREE		
0.0 – 0.3		GRASS AND ROOTS
0.3 – 1.3		LIGHT BROWN, SILTY FINE TO MEDIUM SAND WITH ROOTS AND GRAVEL (MEDIUM DENSE TO DENSE, MOIST) (**FILL**)
1.3– 8.5		BROWN TO GRAY, SILTY FINE TO COARSE SAND WITH GRAVEL, CEMENTED CHUNKS OF SILT WITH FINE SAND, COBBLES, WOOD DEBRIS, AND TRACE GARBAGE (MEDIUM DENSE TO DENSE, MOIST) (**FILL**)
		SAMPLE WAS COLLECTED AT 6.5 AND 8.5 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 8.5 FEET ON 8/31/16
TEST PIT FOUR		
0.0 – 0.3		GRASS AND ROOTS
0.3 – 1.3		LIGHT BROWN, SILTY FINE TO MEDIUM SAND WITH GRAVEL, AND ROOTS (MEDIUM DENSE TO DENSE, MOIST) (**FILL**)
1.3– 4.0		BROWN, GRAVELLY FINE TO COARSE SAND WITH SILT, CONCRETE RUBBLE, AND WOOD DEBRIS (MEDIUM DENSE TO DENSE, MOIST) (**FILL**)
4.0 – 6.0		GRAY, WELL CEMENTED SILTY FINE TO MEDIUM SAND WITH GRAVEL, COBBLES, AND TRACE WOOD DEBRIS (DENSE TO VERY DENSE, MOIST) (**FILL**)
		SAMPLES WERE COLLECTED AT 4.5, 5.0, 6.0 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 6.0 FEET ON 8/31/16

LOG OF EXPLORATION

DEPTH (FEET)	USC	SOIL DESCRIPTION
TEST PIT FIVE		
0.0 – 0.2		GRASS AND ROOTS
0.2 – 1.2		LIGHT BROWN, SILTY FINE TO MEDIUM SAND WITH GRAVEL, ROOTS, AND TRACE COBBLES (MEDIUM DENSE TO DENSE, MOIST) (**FILL**)
1.2– 4.0		GRAY TO BROWN, GRAVEL WITH SILTY FINE TO COARSE SAND, WOOD DEBRIS, AND TRACE SILT CLOTH SCRAP (FOUND AT 1.5 FEET) (MEDIUM DENSE TO DENSE, MOIST)(**FILL**)
4.0 – 5.5		GRAY, WELL CEMENTED SILTY FINE TO MEDIUM SAND WITH GRAVEL, TRACE COBBLES, AND ASPHALT GRINDINGS (DENSE TO VERY DENSE, MOIST)(**FILL**)
		SAMPLES WERE COLLECTED AT 3.5 AND 5.5 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 5.5 FEET ON 8/31/16
TEST PIT SIX		
0.0 – 0.2		GRASS AND ROOTS
0.2 – 1.2		LIGHT BROWN, SILTY FINE TO MEDIUM SAND WITH GRAVEL, COBBLES, AND ROOTS (MEDIUM DENSE TO DENSE, MOIST) (**FILL**)
1.2– 4.5		BROWN GRAVEL WITH SILTY FINE TO COARSE SAND, COBBLES, WOOD DEBRIS, AND TRACE GARBAGE (MEDIUM DENSE TO DENSE, MOIST) (**FILL**)
4.5 – 6.0		GRAY TO BROWN, SILTY FINE TO COARSE SAND WITH GRAVEL, COBBLES, TRACE BRICKS AND GARBAGE (DENSE TO VERY DENSE, MOIST) (**FILL**)
		SAMPLE WAS COLLECTED AT 6.0 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS ENCOUNTERED TEST PIT WAS COMPLETED AT 6.0 FEET ON 8/31/16
TEST PIT SEVEN		
0.0 – 0.2		GRASS AND ROOTS
0.2 – 1.5		LIGHT BROWN, SILTY FINE TO MEDIUM SAND WITH GRAVEL AND ROOTS (MEDIUM DENSE TO DENSE, MOIST) (**FILL**)
1.5– 7.0		BROWN TO GRAY, SILTY FINE TO COARSE SAND WITH GRAVEL, COBBLES, WOOD DEBRIS, AND TRACE ASPHALT GRINDINGS (MEDIUM DENSE TO DENSE, MOIST) (**FILL**)
		SAMPLES WERE COLLECTED AT 4.0 AND 7.0 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS ENCOUNTERED TEST PIT WAS COMPLETED AT 7.0 FEET ON 8/31/16

LOG OF EXPLORATION

DEPTH (FEET)	USC	SOIL DESCRIPTION

TEST PIT EIGHT

0.0 – 0.2		GRASS AND ROOTS
0.2 – 2.2		LIGHT BROWN, SILTY FINE TO MEDIUM SAND WITH GRAVEL, COBBLES, ROOTS, AND TRACE METAL SCRAPS AT 1.5 FEET (MEDIUM DENSE TO DENSE, MOIST) (**FILL**)
2.2 – 6.0		GRAY TO BROWN, SILTY FINE TO MEDIUM SAND WITH GRAVEL, COBBLES, WOOD DEBRIS, AND TRACE SILT CLOTH (MEDIUM DENSE TO DENSE, MOIST) (**FILL**)

SAMPLES WERE COLLECTED AT 4.5 AND 6.0 FEET
GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED
TEST PIT CAVING WAS ENCOUNTERED
TEST PIT WAS COMPLETED AT 6.0 FEET ON 8/31/16

TEST PIT NINE

0.0 – 2.0		GRAVEL SURFACING AND 2- TO 4-INCH QUARRY SPALLS (DENSE TO VERY DENSE, MOIST) (**FILL**)
2.0 – 5.5		BROWN, SILTY FINE TO MEDIUM SAND WITH GRAVEL, CONCRETE RUBBLE, ASPHALT GRINDINGS, AND TRACE IRON-OXIDE WEATHERING (DENSE TO VERY DENSE, MOIST) (**FILL**)

SAMPLES WERE COLLECTED AT 3.0 AND 5.5 FEET
GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED
TEST PIT CAVING WAS ENCOUNTERED
TEST PIT WAS COMPLETED AT 5.5 FEET ON 8/31/16

TEST PIT TEN

0.0 – 0.2		GRASS AND ROOTS
0.2 – 1.5		LIGHT BROWN, SILTY FINE TO MEDIUM SAND WITH GRAVEL, ROOTS, AND TRACE COBBLES (MEDIUM DENSE TO DENSE, MOIST) (**FILL**)
1.5– 4.5		GRAY TO BROWN, SILTY FINE TO COARSE SAND WITH GRAVEL, COBBLES, GARBAGE, METAL SCRAPS, AND WOOD DEBRIS (MEDIUM DENSE TO DENSE, MOIST) (**FILL**)

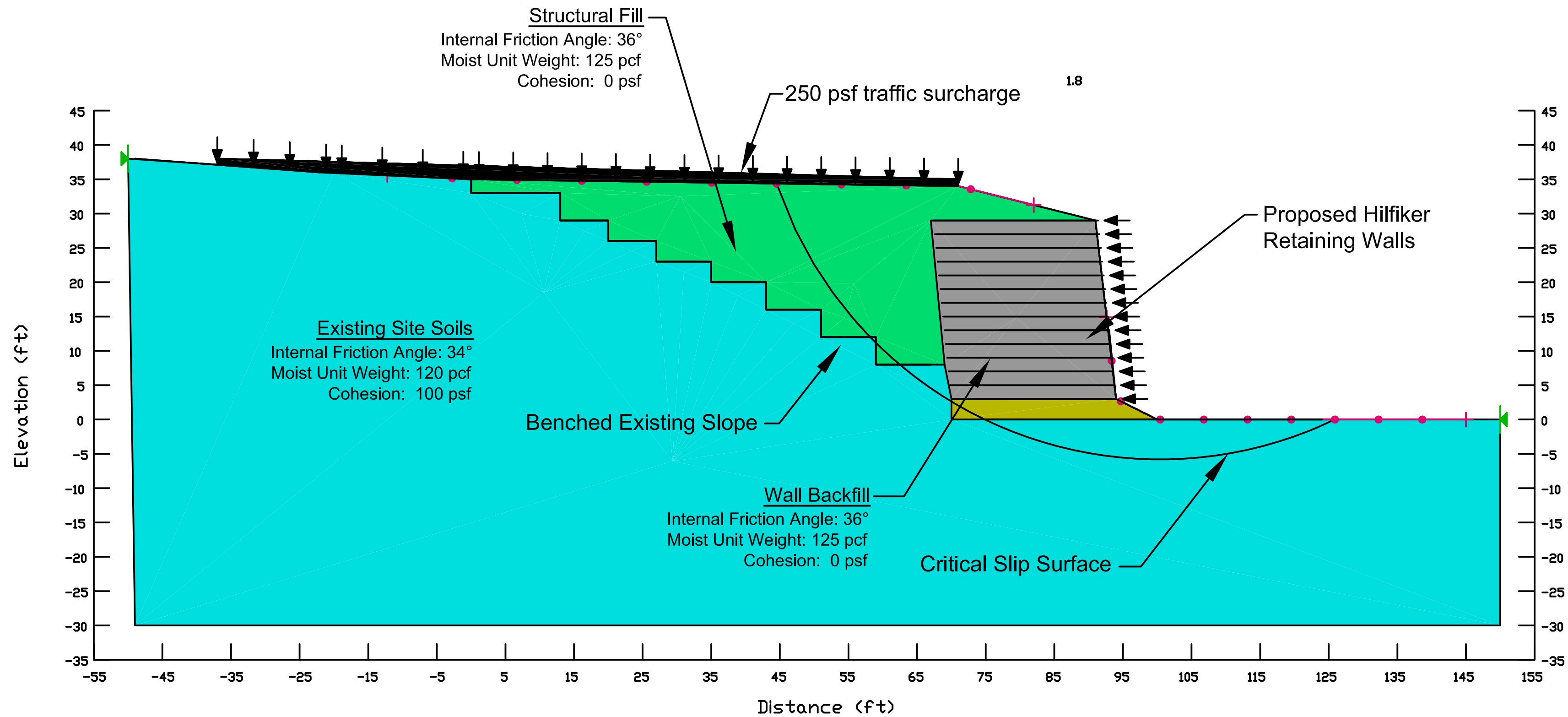
SAMPLE WAS COLLECTED AT 4.5 FEET
GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED
TEST PIT CAVING WAS ENCOUNTERED
TEST PIT WAS COMPLETED AT 4.5 FEET ON 8/31/16

LOG OF EXPLORATION

DEPTH (FEET)	USC	SOIL DESCRIPTION
TEST PIT ELEVEN		
0.0 – 0.2		GRAVEL SURFACING
0.2 – 1.0		LIGHT BROWN, SILTY FINE TO MEDIUM SAND WITH GRAVEL AND TRACE ROOTS (MEDIUM DENSE TO DENSE, MOIST) (**FILL**)
1.0– 4.0		BROWN GRAVEL WITH SILTY FINE TO COARSE SAND, COBBLES, WOOD DEBRIS, AND TRACE GARBAGE (MEDIUM DENSE TO DENSE, MOIST) (**FILL**)
		SAMPLE WAS COLLECTED AT 6.0 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS ENCOUNTERED TEST PIT WAS COMPLETED AT 6.0 FEET ON 8/31/16

Slope Stability Analysis- Proposed Single-Tier Static Conditions

Bishop most critical surface with minimum FOS = 1.8



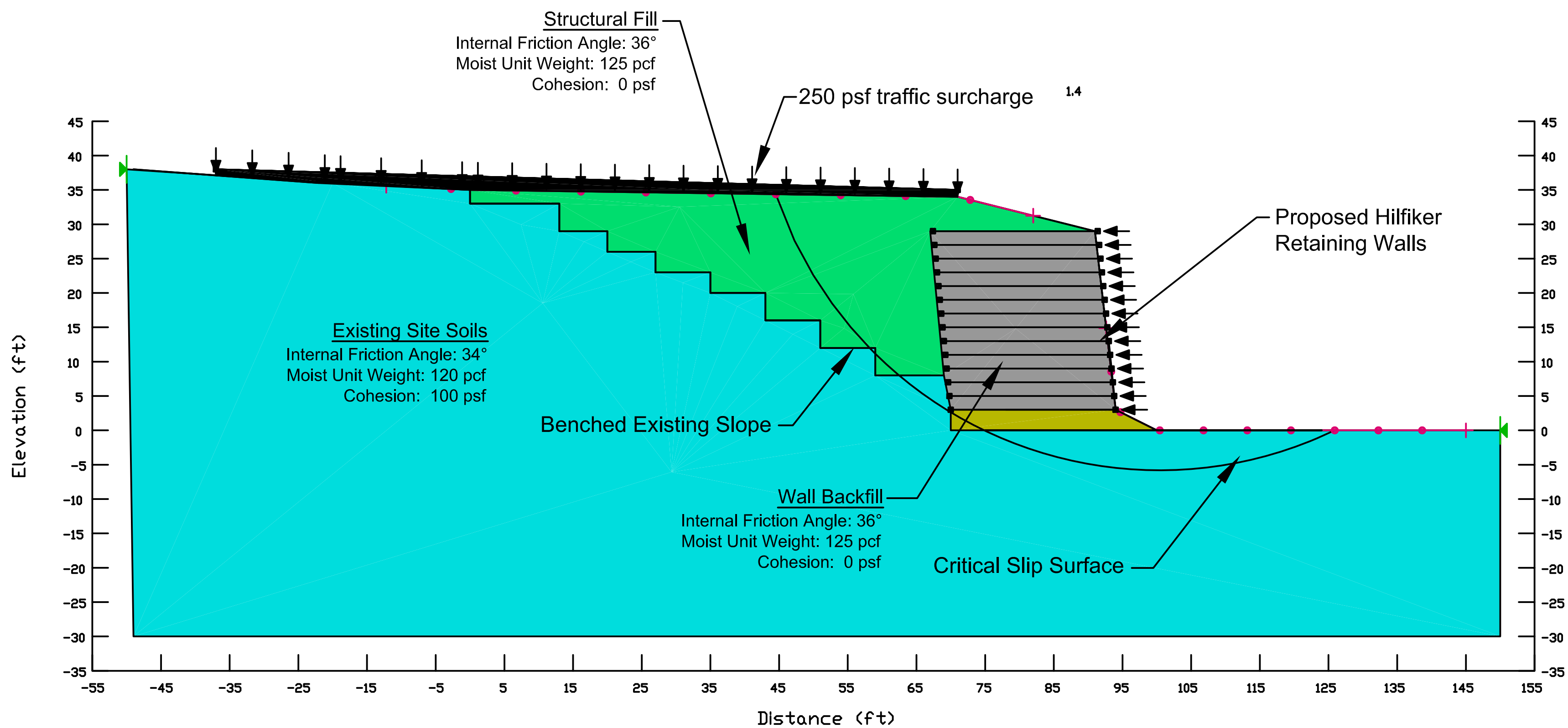
NOTES: Not to scale
Elevations are Approximate

2016 NGA Project Folders\969616 Union Hill Self-Storage\Drafting\Slope Stability Runs.dwg

Project Number			No.	Date	Revision	By	CK
969616	Union Hill Self-Storage Hilfiker Wall Single-Tier Static Slope Stability Analysis	NELSON GEOTECHNICAL ASSOCIATES, INC. GEOTECHNICAL ENGINEERS & GEOLOGISTS 17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax 481-2510 Snohomish County (425) 337-1669 Wenatchee/Chelan (509) 784-2756 www.nelsongeotech.com	1	9/20/16	Original	LSB	KMS
Figure 9							

Slope Stability Analysis- Proposed Single-Tier Seismic Conditions

Bishop most critical surface with minimum FOS = 1.4
with a seismic coefficient of ground acceleration = 0.15g



NOTES: Not to scale
Elevations are Approximate

Project Number	Union Hill Self-Storage Hilfiker Wall Single-Tier Seismic Slope Stability Analysis	Nelson Geotechnical Associates, Inc. Geotechnical Engineers & Geologists	No.	Date	Revision	By	CK
969616		17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax 481-2510 Snohomish County (425) 337-1669 Wenatchee/Chelan (509) 784-2756 www.nelsongeotech.com	1	9/20/16	Original	LSB	KMS
Figure 10							

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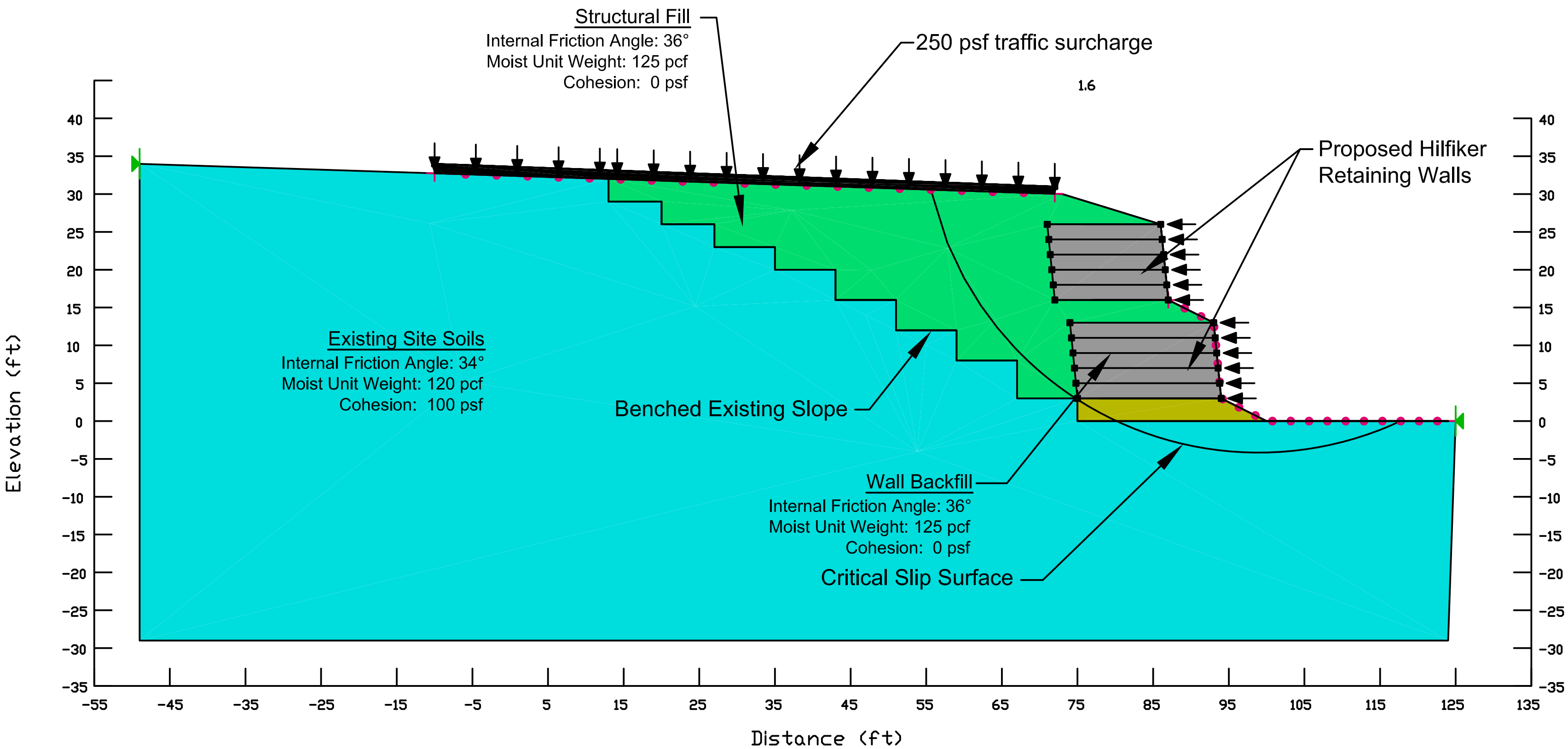
Slope Stability Analysis- Proposed Tiered Walls with Traffic Surcharge Static Conditions

Bishop most critical surface with minimum FOS = 1.6



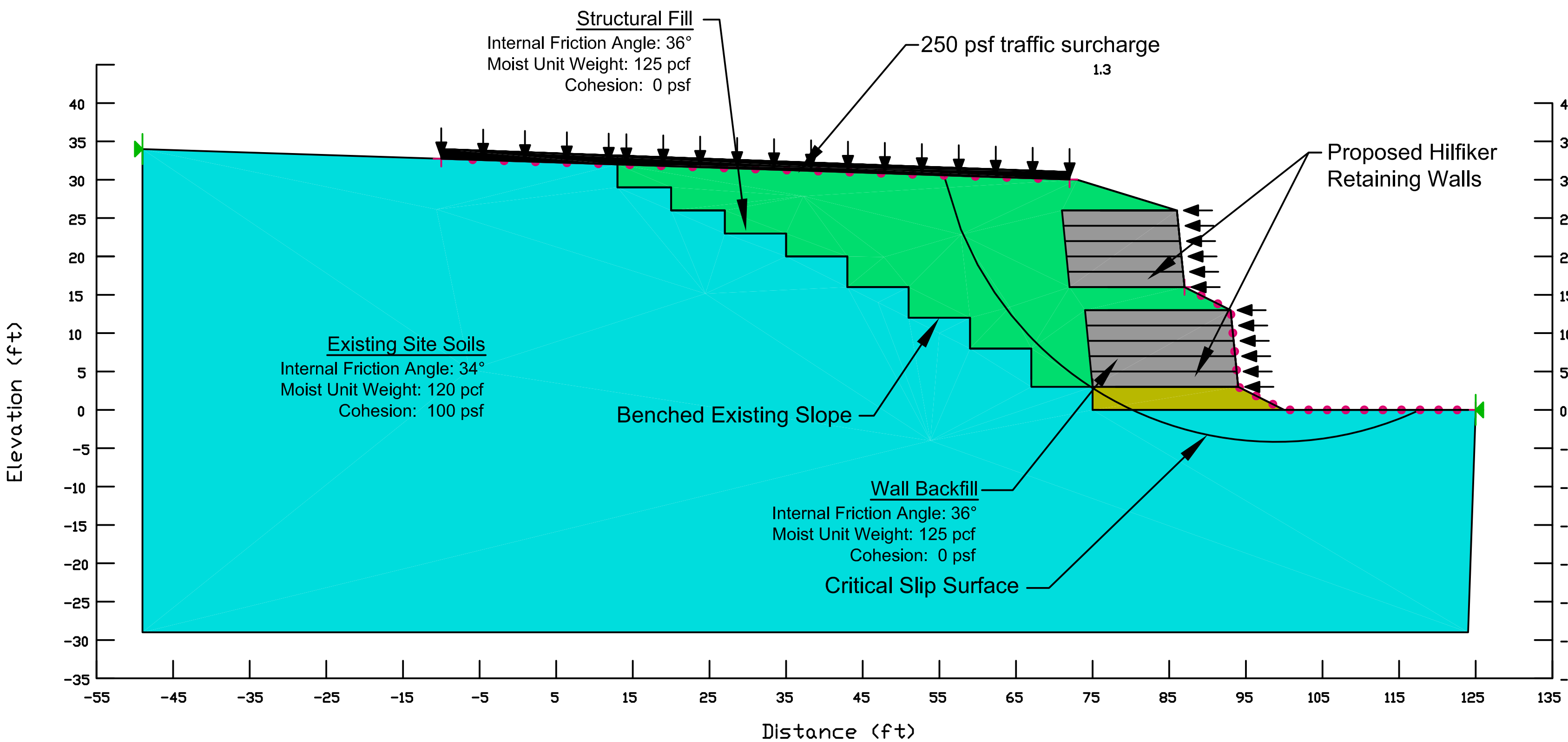
NOTES: Not to scale
Elevations are Approximate

2016 NGA Project Folders\969616 Union Hill Self-Storage\Drafting\Slope Stability Runs.dwg

Project Number	Union Hill Self-Storage	NELSON GEOTECHNICAL ASSOCIATES, INC.	No.	Date	Revision	By	CK
969616	Hilfiker Wall Tiered Walls with Traffic Surcharge Static	GEOTECHNICAL ENGINEERS & GEOLOGISTS	1	9/20/16	Original	LSB	KMS
Figure 11	Slope Stability Analysis	17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax 481-2510; Snohomish County (425) 337-1669 Wenatchee/Chelan (509) 784-2756 www.nelsongeotech.com					

Slope Stability Analysis- Proposed Tiered Walls with Traffic Surcharge Seismic Conditions

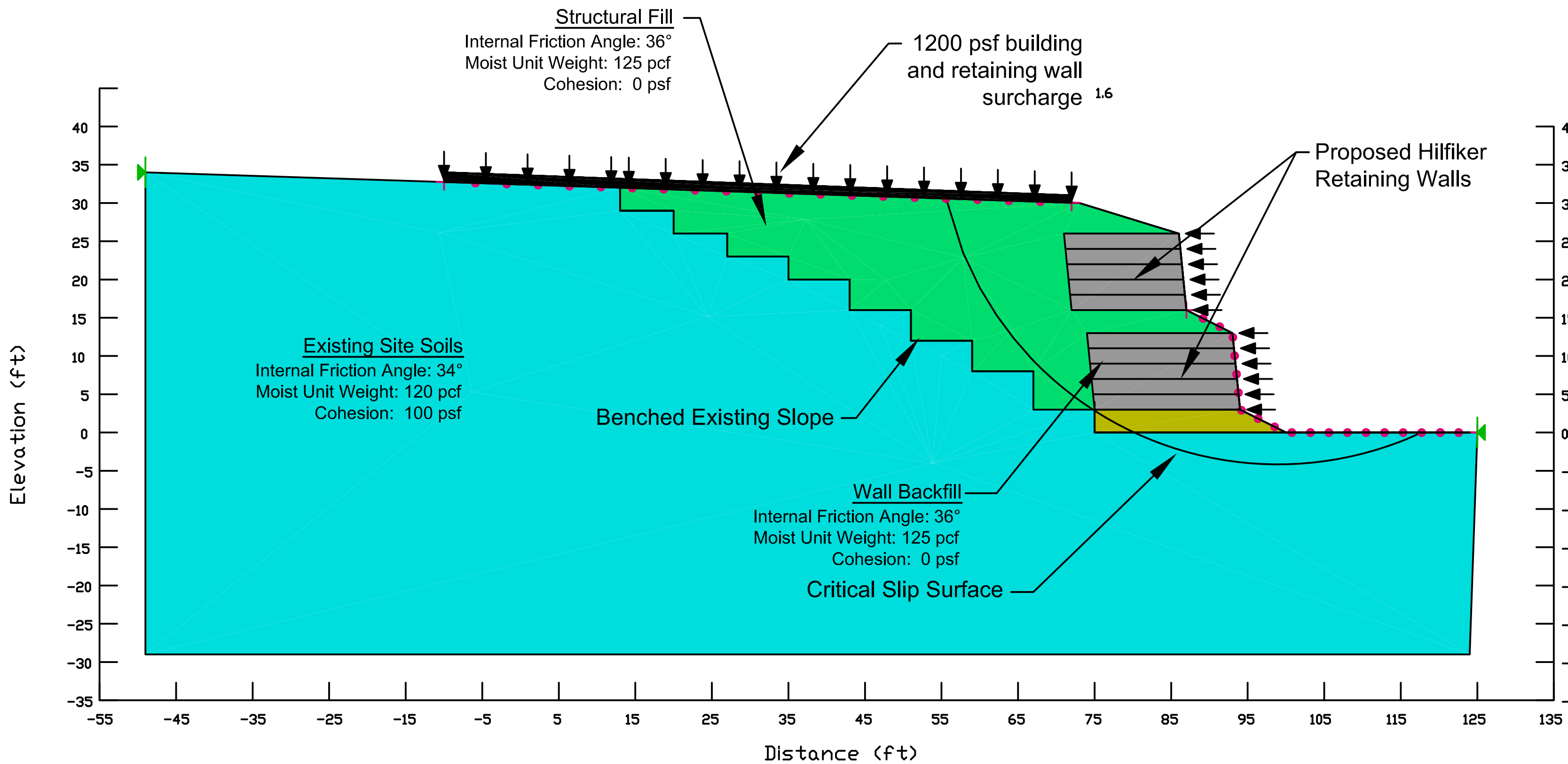
Bishop most critical surface with minimum FOS = 1.3
with a seismic coefficient of ground acceleration = 0.15g



NOTES: Not to scale
Elevations are Approximate

Project Number	Union Hill Self-Storage Hilfiker Wall Tiered Walls with Traffic Surcharge Seismic Slope Stability Analysis	NELSON GEOTECHNICAL ASSOCIATES, INC. GEOTECHNICAL ENGINEERS & GEOLOGISTS	No.	Date	Revision	By	CK
969616		17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax 481-2510 Snohomish County (425) 337-1669 Wenatchee/Chelan (509) 784-2756 www.nelsongeotech.com	1	9/20/16	Original	LSB	KMS
Figure 12							

Slope Stability Analysis- Proposed Tiered Walls with Building and Retaining Wall Surcharge Static Conditions
Bishop most critical surface with minimum FOS = 1.6



NOTES: Not to scale
Elevations are Approximate

Project Number	Union Hill Self-Storage Hilfiker Wall Tiered Walls with Building and Ret. Wall Surcharge Static Slope Stability Analysis	Nelson Geotechnical Associates, Inc. Geotechnical Engineers & Geologists	No.	Date	Revision	By	CK
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Figure 13							

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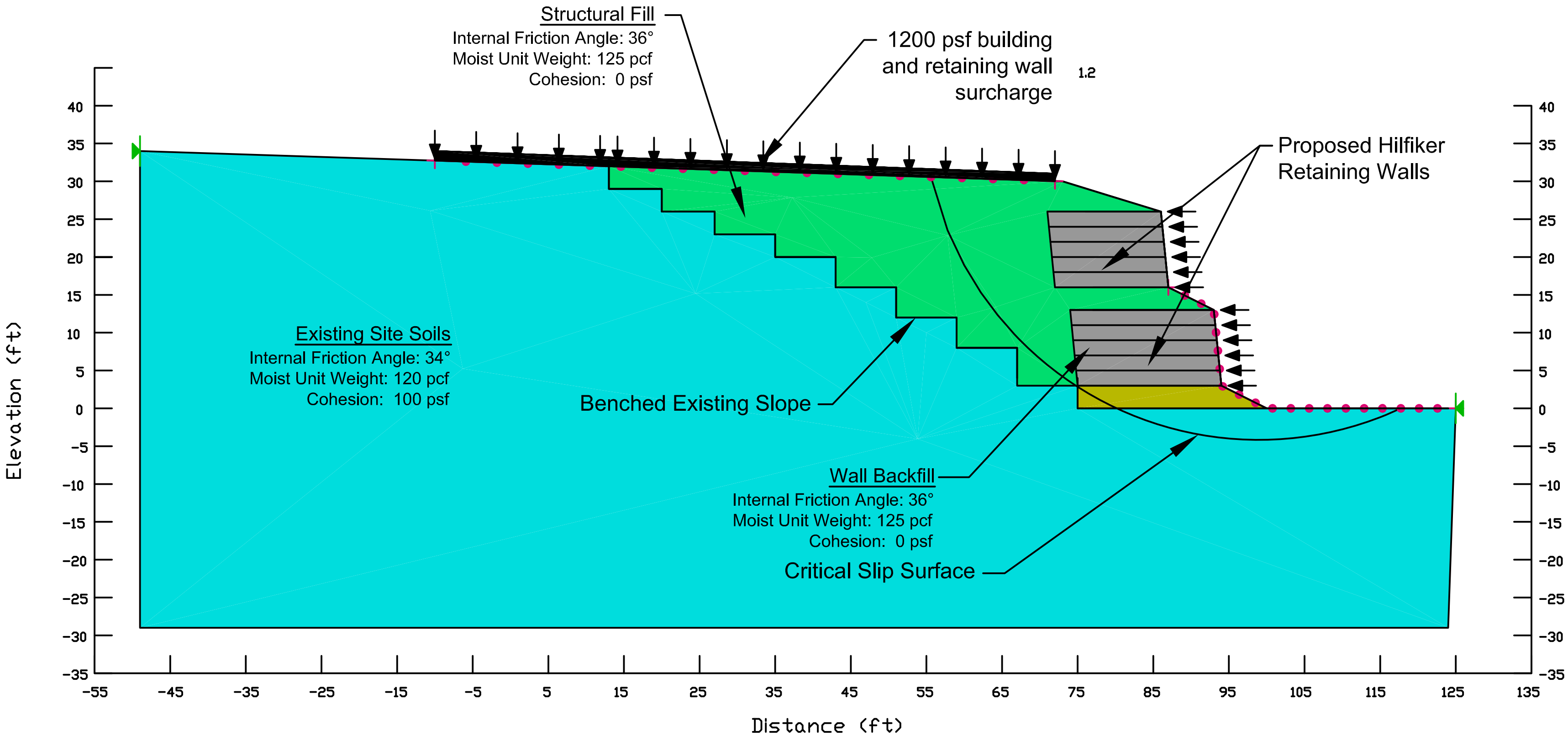
Slope Stability Analysis- Proposed Tiered Walls with Building and Retaining Wall Surcharge Seismic Conditions

Bishop most critical surface with minimum FOS = 1.2
with a seismic coefficient of ground acceleration = 0.15g



NOTES: Not to scale
Elevations are Approximate

Project Number	Union Hill Self-Storage Hilfiker Wall Tiered Walls with Building and Ret. Wall Surcharge Seismic Slope Stability Analysis
969616	
Figure 14	

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GEOTECHNICAL ENGINEERS & GEOLOGISTS
17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax 481-2510
Snohomish County (425) 337-1669
Wenatchee/Chelan (509) 784-2756
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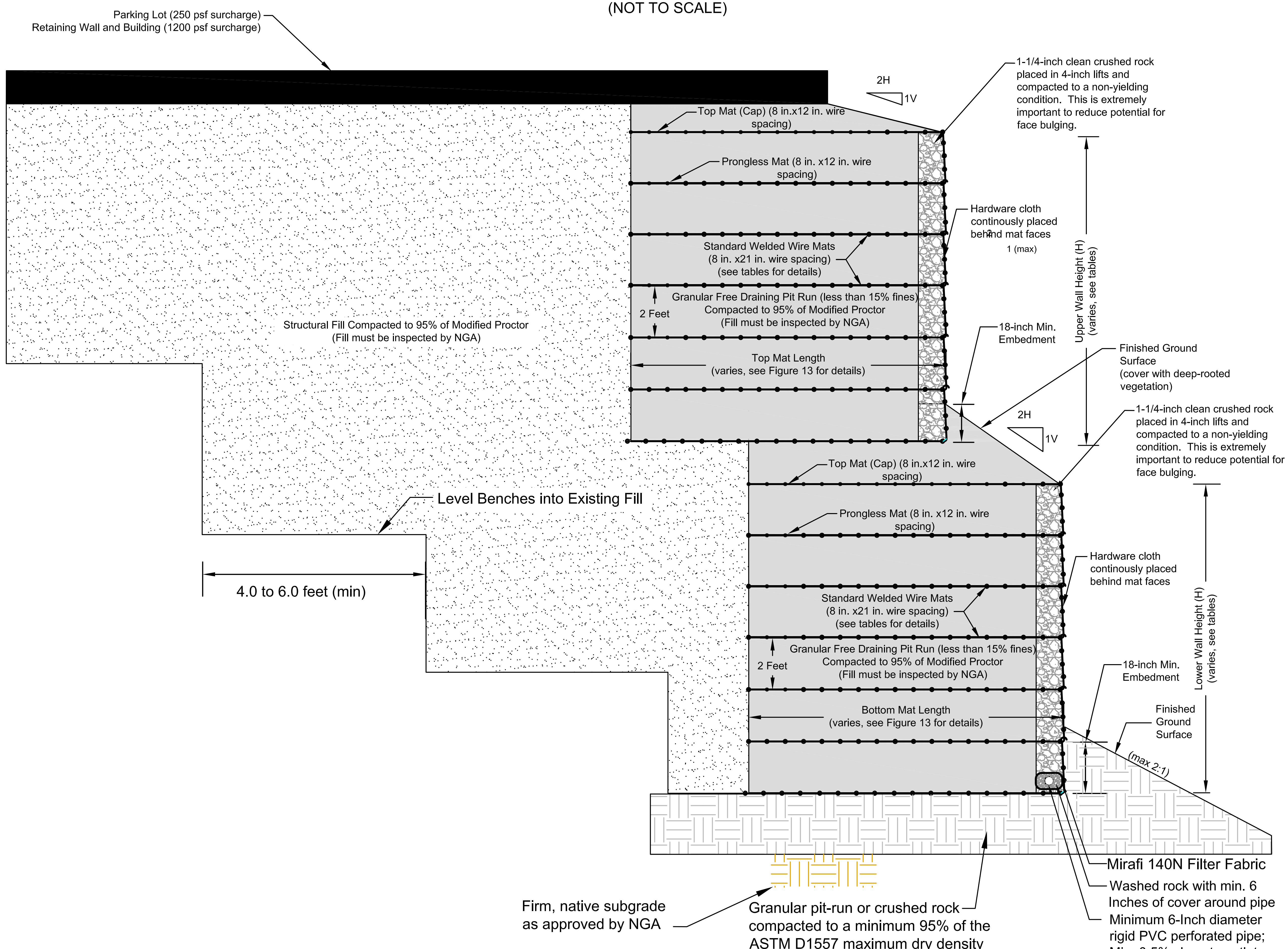
Hilfiker Welded Wire Retaining Wall

(NOT TO SCALE)



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Geotechnical Engineers & Geologists
17311-135th Ave. NE, A-500
Woodinville, WA 98072
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Snohomish County (425) 337-1669
Wenatchee/Chelan (509) 784-2756
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Union Hill Self-Storage
Welded Wire Retaining Wall
Cross-Section

Project Number
969616

Figure 15

Hilfiker Welded Wire Retaining Wall - Mat Details

(NOT TO SCALE)

Single-Tier Walls - Northern Walls

Total Wall Height (ft.)	Wire Size Long x Trans	Wire Spacing Long x Trans	Number of Mats	Mat Length (ft.)	Reinforcing Layer Number and Elevation Above Wall Subgrade (ft.)													
					1	2	3	4	5	6	7	8	9	10	11	12	13	14
4	W7.0 x W4.0	8 in x 21 in	1	6	0													
	W4.5 x W3.5	8 in x 12 in*	2	6		2	4											
6	W7.0 x W4.0	8 in x 21 in	2	6	0	2												
	W4.5 x W3.5	8 in x 12 in*	2	6			4	6										
8	W7.0 x W4.0	8 in x 21 in	3	8	0	2	4											
	W4.5 x W3.5	8 in x 12 in*	2	8				6	8									
10	W7.0 x W4.0	8 in x 21 in	4	10.5	0	2	4	6										
	W4.5 x W3.5	8 in x 12 in*	2	10.5					8	10								
12	W7.0 x W4.0	8 in x 21 in	5	12.5	0	2	4	6	8									
	W4.5 x W3.5	8 in x 12 in*	2	12.5						10	12							
14	W9.5 x W4.0	8 in x 21 in	6	14	0	2	4	6	8	10								
	W9.5 x W4.0	8 in x 12 in*	2	14							12	14						
16	W9.5 x W4.0	8 in x 21 in	7	14	0	2	4	6	8	10	12							
	W9.5 x W4.0	8 in x 12 in*	2	14								14	16					
18	W9.5 x W4.0	8 in x 21 in	8	15	0	2	4	6	8	10	12	14						
	W9.5 x W4.0	8 in x 12 in*	2	15									16	18				
20	W9.5 x W4.0	8 in x 21 in	9	17	0	2	4	6	8	10	12	14	16					
	W9.5 x W4.0	8 in x 12 in*	2	17										18	20			
22	W9.5 x W4.0	8 in x 21 in	10	19	0	2	4	6	8	10	12	14	16	18				
	W9.5 x W4.0	8 in x 12 in*	2	19											20	22		
24	W9.5 x W4.0	8 in x 21 in	11	21	0	2	4	6	8	10	12	14	16	18	20			
	W9.5 x W4.0	8 in x 12 in*	2	21												22	24	24
26	W9.5 x W4.0	8 in x 21 in	12	24	0	2	4	6	8	10	12	14	16	18	20	22		
	W9.5 x W4.0	8 in x 12 in*	2	24													24	26

* Prongless and top mat - See illustrations.

Two-Tier Walls - Western Walls

Upper Tier

Total Wall Height (ft.)	Wire Size Long x Trans	Wire Spacing Long x Trans	Number of Mats	Mat Length (ft.)	Reinforcing Layer Number and Elevation Above Wall Subgrade (ft.)						
					1	2	3	4	5	6	7
4	W7.0 x W4.0	8 in x 21 in	1	15.75	0						
	W7.0 x W4.0	8 in x 12 in*	2	15.75		2	4				
6	W7.0 x W4.0	8 in x 21 in	2	15.75	0	2					
	W7.0 x W4.0	8 in x 12 in*	2	15.75			4	6			
8	W7.0 x W4.0	8 in x 21 in	3	15.75	0	2	4				
	W7.0 x W4.0	8 in x 12 in*	2	15.75				6	8		
10	W7.0 x W4.0	8 in x 21 in	4	15.75	0	2	4	6			
	W7.0 x W4.0	8 in x 12 in*	2	15.75					8	10	
12	W7.0 x W4.0	8 in x 21 in	5	15.75	0	2	4	6	8		
	W7.0 x W4.0	8 in x 12 in*	2	15.75						10	12

Lower Tier

Total Wall Height (ft.)	Wire Size Long x Trans	Wire Spacing Long x Trans	Number of Mats	Mat Length (ft.)	Reinforcing Layer Number and Elevation Above Wall Subgrade (ft.)						
					1	2	3	4	5	6	7
4	W9.5 x W4.0	8 in x 21 in	1	21	0						
	W9.5 x W4.0	8 in x 12 in*	2	21		2	4				
6	W9.5 x W4.0	8 in x 21 in	2	21	0	2					
	W9.5 x W4.0	8 in x 12 in*	2	21			4	6			
8	W9.5 x W4.0	8 in x 21 in	3	21	0	2	4				
	W9.5 x W4.0	8 in x 12 in*	2	21				6	8		
10	W9.5 x W4.0	8 in x 21 in	4	21	0	2	4	6			
	W9.5 x W4.0	8 in x 12 in*	2	21					8	10	
12	W9.5 x W4.0	8 in x 21 in	5	21	0	2	4	6	8		
	W9.5 x W4.0	8 in x 12 in*	2	21						10	12

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NGA
GEOTECHNICAL ENGINEERS & GEOLOGISTS
17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax 481-2510
Snohomish County (425) 337-1669
Wenatchee/Chelan (509) 784-2756
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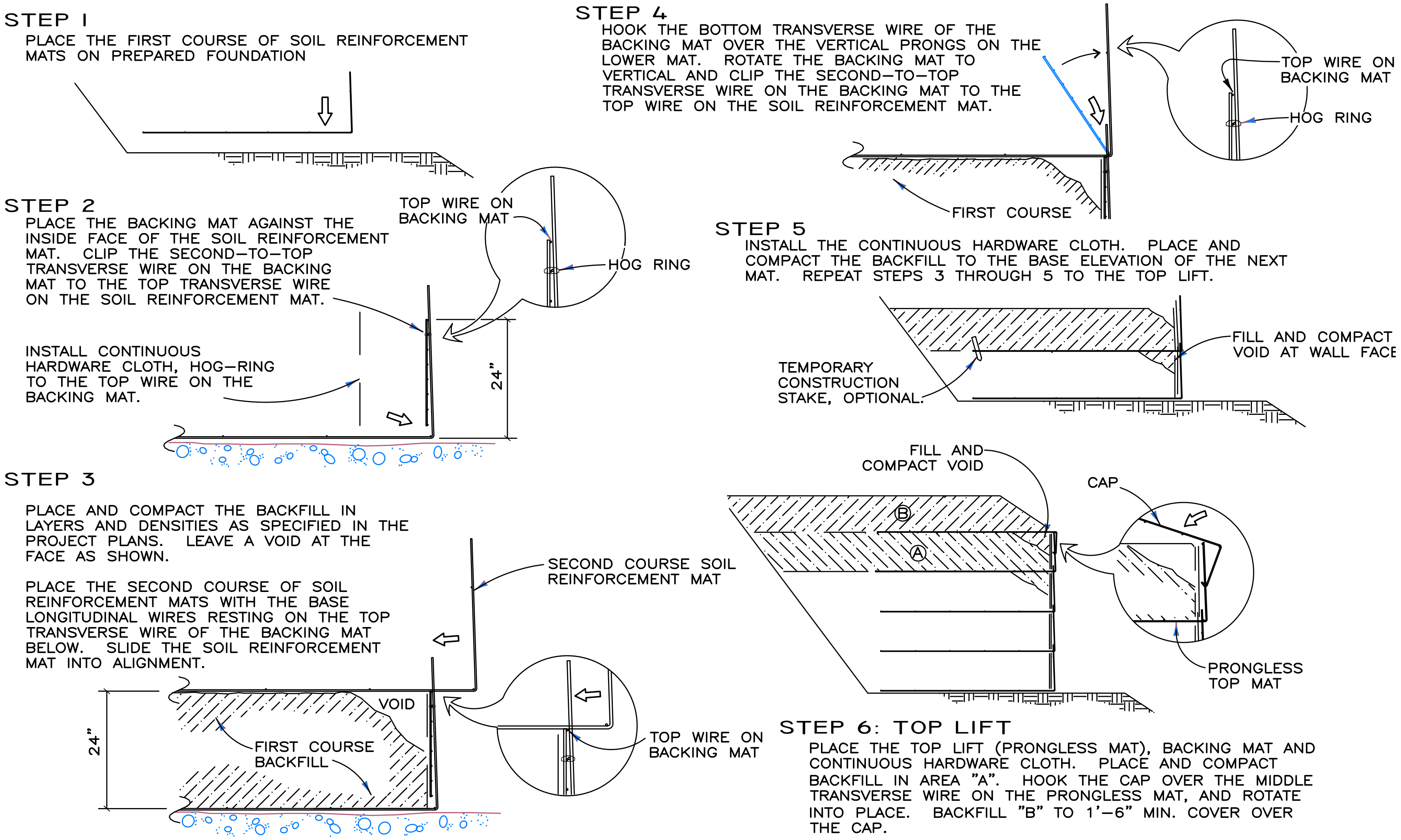
Union Hill Self Storage
Hilfiker Mat Design Tables

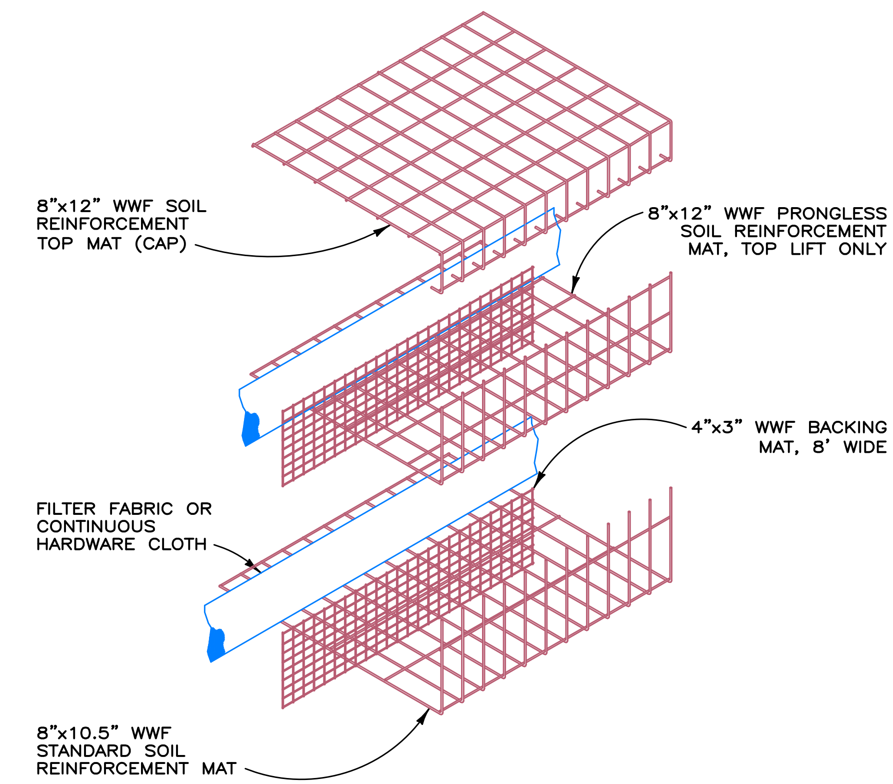
Project Number
969616

Figure 16

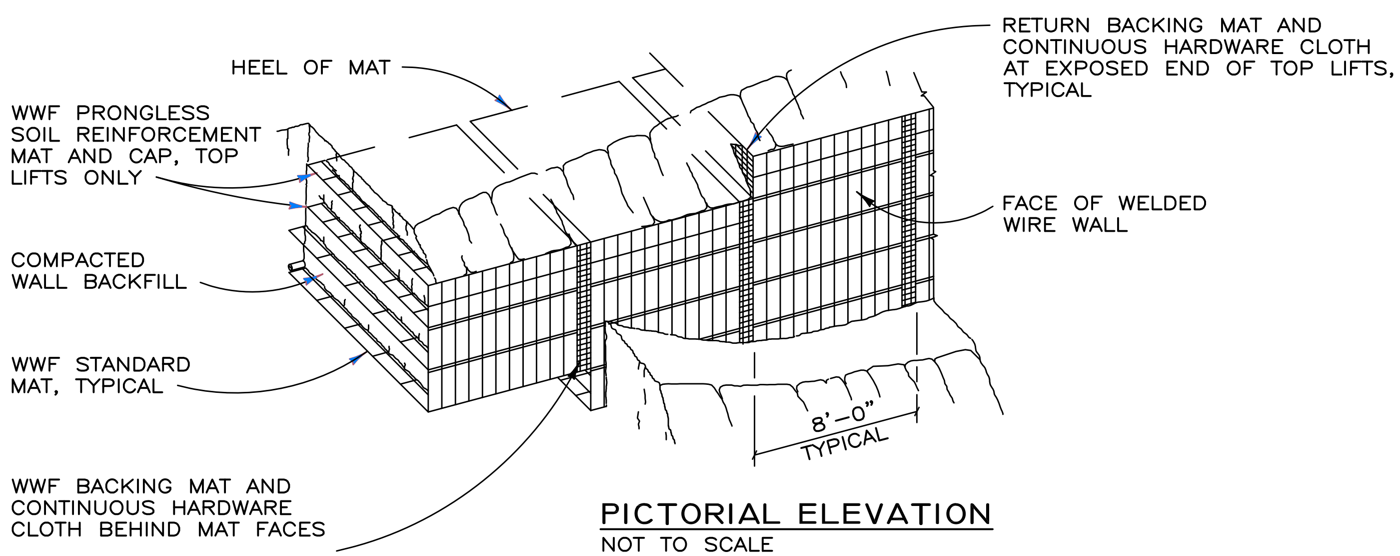
Standard Hilfiker Wall details provided by Hilfiker Retaining Walls.



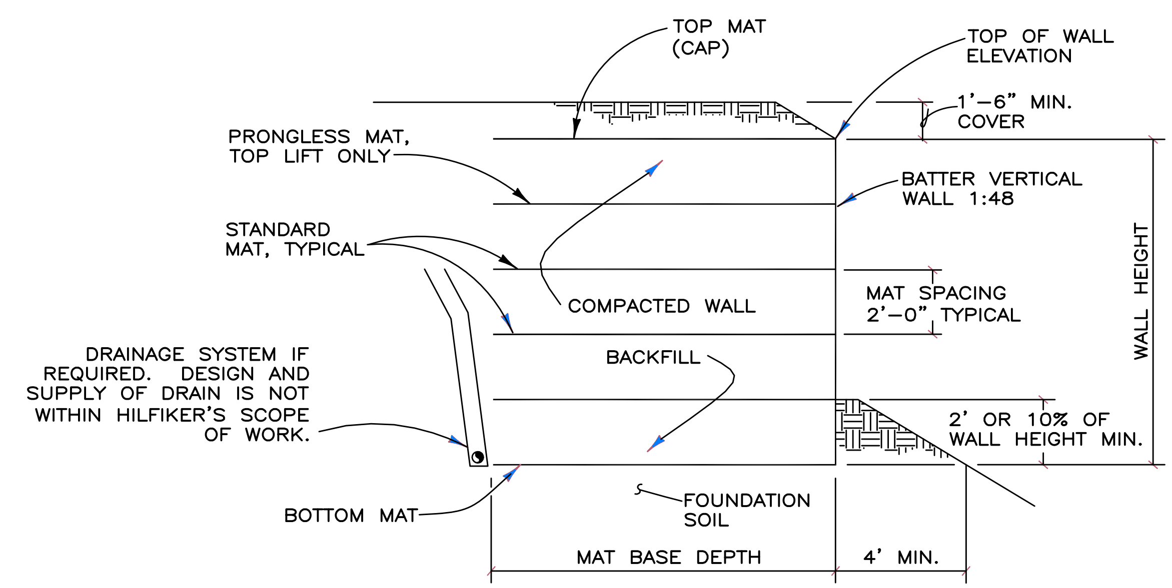
CONSTRUCTION SEQUENCE
NOT TO SCALE



WALL COMPONENTS
NOT TO SCALE



PICTORIAL ELEVATION
NOT TO SCALE



TYPICAL SECTION*
NOT TO SCALE
* Refer to detailed cross-section on Figure 3

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17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax 481-2510

Snohomish County (425) 337-1669
Wenatchee/ Chelan (509) 665-7696
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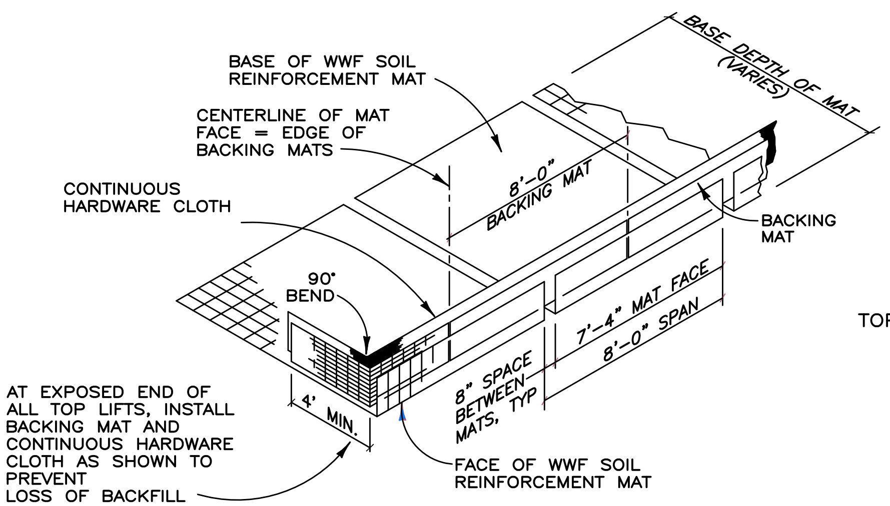
Union Hill Self-Storage
Welded Wire Retaining Wall
Construction Details

Project Number
969616

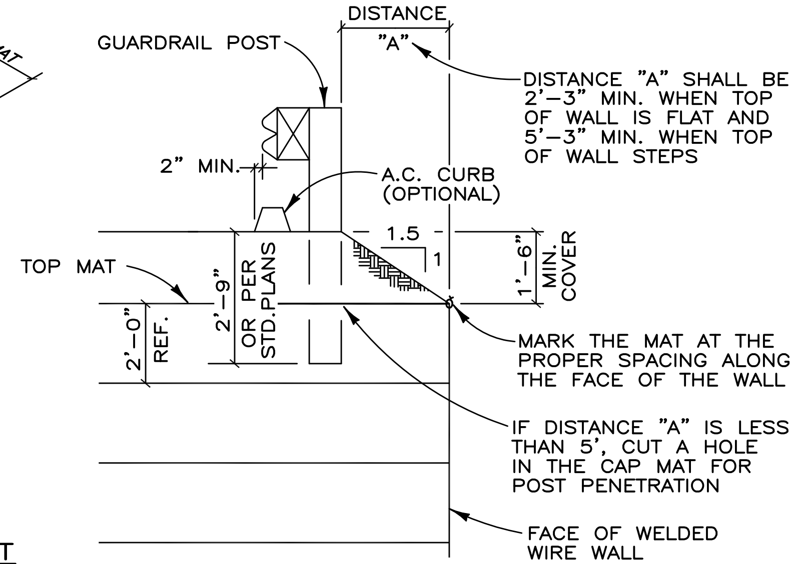
Figure 17

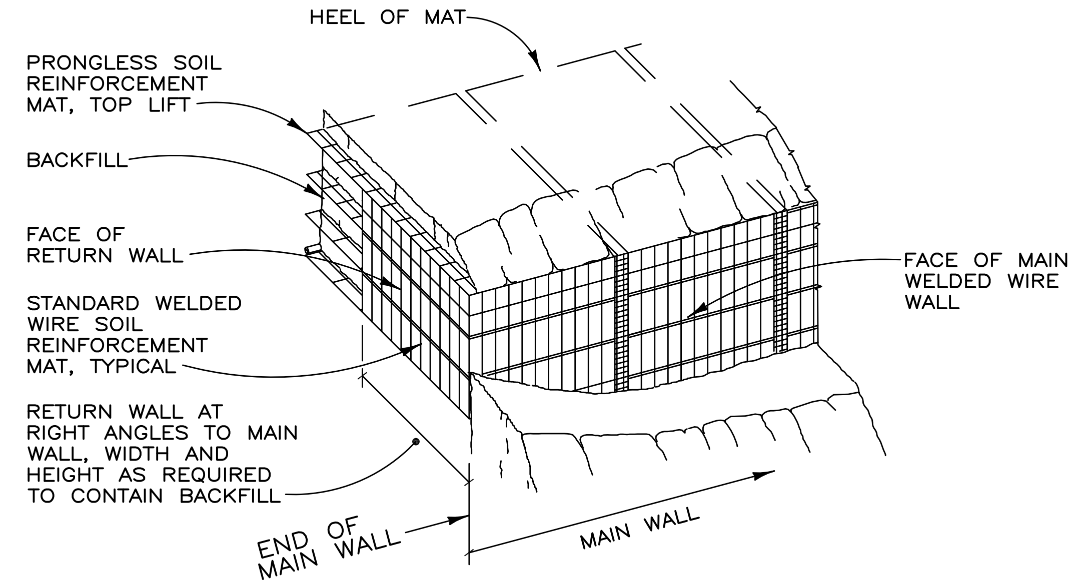


ISOMETRIC VIEW

WELDED WIRE WALL COMPONENTS WITH RETURN MAT

NOT TO SCALE

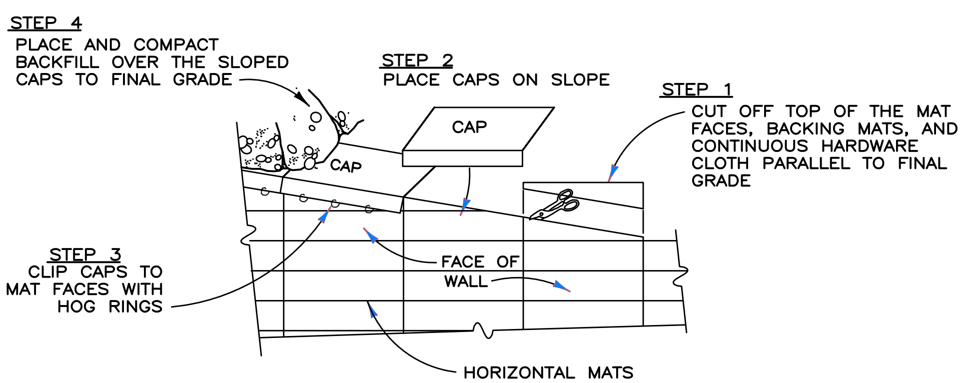




PICTORIAL

RETURN WALL DETAIL

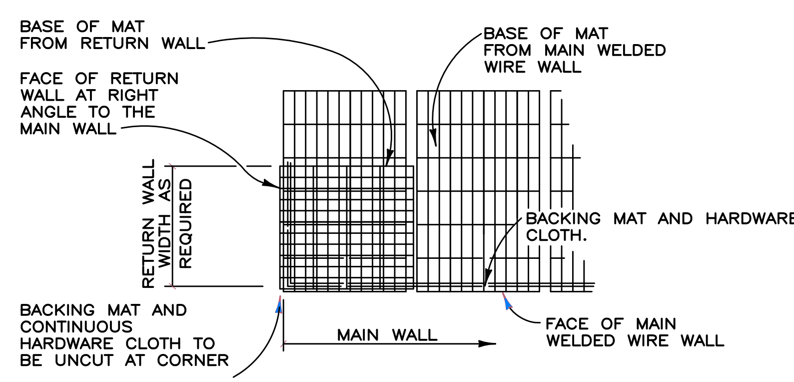
NOT TO SCALE



PICTORIAL ELEVATION

SLOPED CAP MAT DETAIL

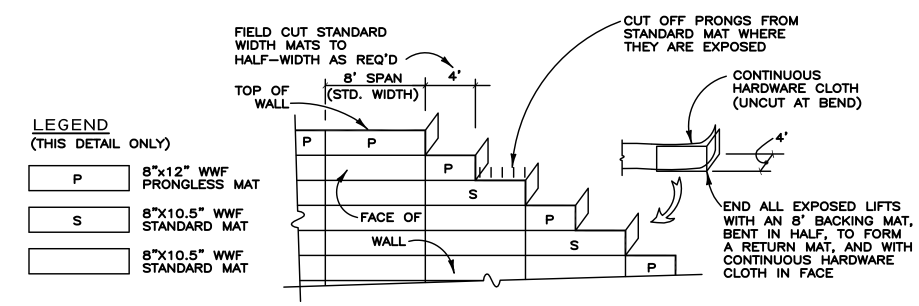
NOT TO SCALE



PLAN VIEW

RETURN WALL DETAIL

NOT TO SCALE



RETURN MATS AND TOP OF WALL DETAIL

NOT TO SCALE

Standard Hilfiker Wall details provided by Hilfiker Retaining Walls.

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17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax 481-2510
Snohomish County (425) 337-1669
Wenatchee/ Chelan (509) 665-7696
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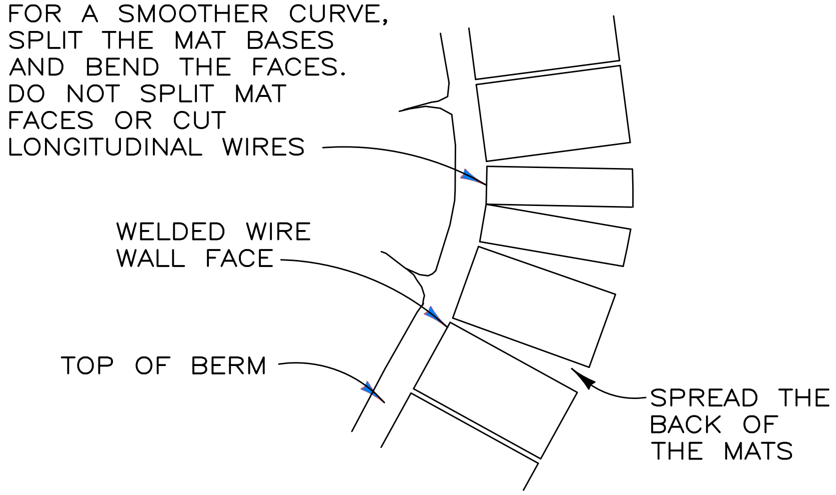
Union Hill Self Storage
Welded Wire Retaining Wall
Construction Details

Project Number
969616

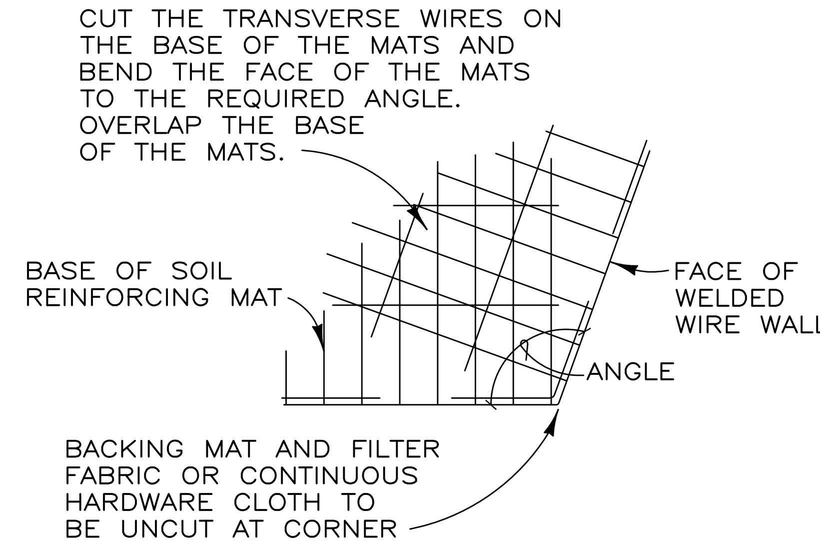
Figure 18



PLAN VIEW

CONCAVE CURVE

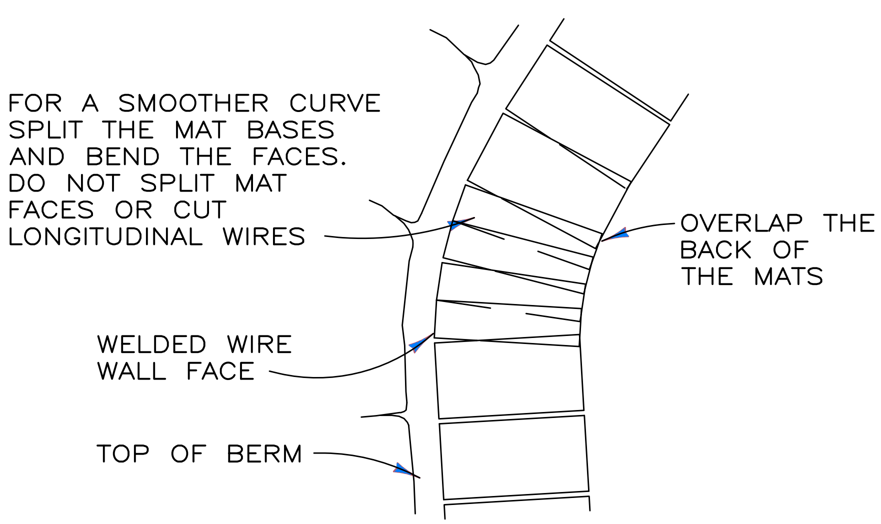
NOT TO SCALE



PLAN VIEW

OBTUSE CONVEX ANGLE

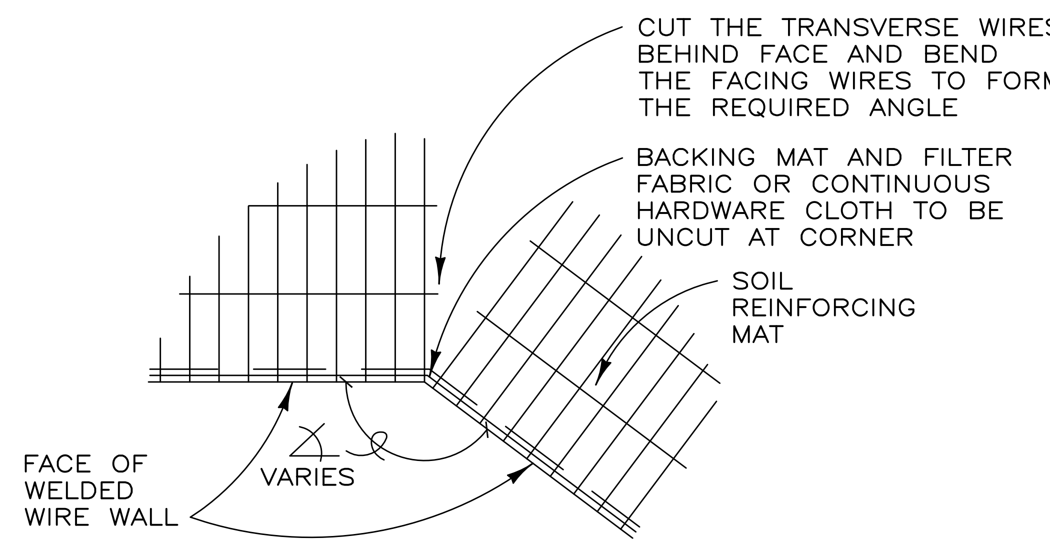
NOT TO SCALE



PLAN VIEW

CONVEX CURVE

NOT TO SCALE



PLAN VIEW

CONCAVE ANGLE DETAIL

NOT TO SCALE

Standard Hilfiker Wall details provided by Hilfiker Retaining Walls.

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GEOTECHNICAL ENGINEERS & GEOLOGISTS
17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax 481-2510
Snohomish County (425) 337-1669
Wenatchee/ Chelan (509) 665-7696
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Union Hill Self Storage
Welded Wire Retaining Wall
Construction Details

Project Number
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Figure 19

SPECIFICATIONS FOR REINFORCED WALL

General
1. The contractor shall have an approved set of plans and specifications on site at all times during the construction of the wall. The wall layout is the responsibility of the contractor.
2. Nelson Geotechnical Associates (NGA) should observe and monitor the construction of the wall.
3. Mirafi geogrid 5XT or equivalent shall be used for this project. All geogrid and facing materials shall be approved by NGA prior to installation.
4. The contractor may use longer geogrid lengths than the design sections for ease of construction. The geogrid lengths may not be shorter unless approved by NGA.

Subgrade Preparation
1. The ground should be prepared by removing surficial organics and loose soil to expose competent native soils as approved by the NGA.
2. A generally level bench with a minimum width equal to the design length of the geogrid is required for placement of the reinforced fill.
3. The excavation shall be cleaned of all excess material and protected, as necessary, from construction traffic to maintain the integrity of the subgrade.
4. The base of the excavation should be deep enough to satisfy a minimum embedment of 1.0 feet.

Geogrid Placement
1. The reinforcement shall be rolled out, cut to length, and laid at the proper elevation, location, and orientation. Orientation of the reinforcement is of extreme importance since geogrids vary in strength with roll direction. The contractor shall be responsible for the correct orientation.
2. Geogrid shall be placed at the location and elevations shown on the plans. The geogrid length is measured from the back of the block.
3. Prior to placing the fill, the geogrid shall be pulled to remove the slack and stretched by hand until taut and free of wrinkles.

Fill Placement
1. Structural fill, consisting of granular import soils, would then be placed upon the subgrade and geogrid. If larger rock is used in the fill, additional layers of geogrid may need to be used in the reinforcement. The contractor shall prevent damage to the geogrid by placing the first lift of structural fill with at least a 1-foot thickness. NGA shall approve the material placed in the reinforced zone, before placement.
2. Structural fill should have parameters equal to or better than those stated for the reinforced wall fill below with less then 15 percent passing the number 200 sieve. NGA may allow a higher silt content based on review of the wall design and proposed fill parameters.
3. Soil density tests should be performed as designated by the geotechnical engineer.
4. Fill soils in the wall area shall be compacted to at least 95 percent of the Maximum Dry Density (MDD) as determined by ASTM D-1557.
5. The soil shall be placed in relatively uniform horizontal lifts not exceeding 10 or 12 inches in thickness. The lift thickness shall not exceed the manufacturer's recommended depth for the compactive device used on the project.

Drainage
1. A specific drainage system is shown on the plans. Alternative drains can be used based on conditions found in the field and the material used within the reinforced zone. Changes to the drainage system should be approved by NGA prior to placement.
2. A drainage blanket 12 inches in width should be installed directly behind the keystone block facing and shall consist of 3/4-inch clean crushed rock. All of the drainage materials shall have a fines content no greater than 5 percent passing the number 200 sieve. A 4-inch rigid perforated pipe embedded in a minimum of one foot of pea gravel or washed rock and wrapped with filter fabric should be installed at the bottom of the drainage blanket
3. Surface water shall not be allowed to pond in or near the reinforced fill zone during or after construction.
4. Suitable clean-outs should be installed every 50 feet for future maintenance.

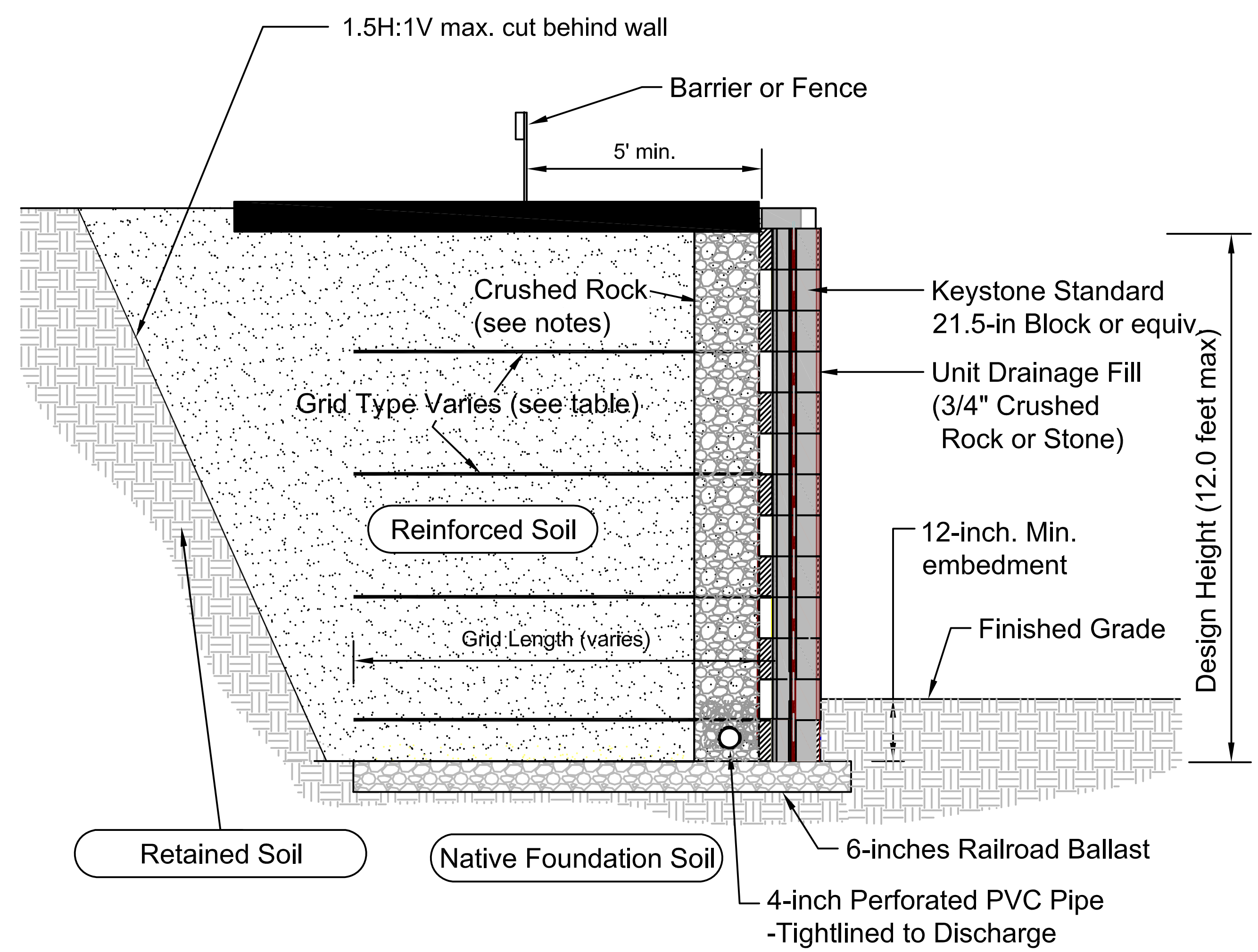
Design Parameters
Reinforced Wall Fill: 34 degrees, 0 PSF, 125 PCF
Retained Backfill: 32 degrees, 0 PSF, 120 PCF
Foundation Soil: 32 degrees, 0 PSF, 120 PCF

External Stability of Wall
Minimum Factor of Safety against Base Sliding: 1.5
Minimum Factor of Safety against Overturning: 2.0
Minimum Factor of Safety against Bearing Capacity: 2.0

Internal Stability of Wall
Minimum Factor of Safety on Geogrid Strength: 1.5
Minimum Factor of Safety on Geogrid Pullout: 1.5
Soil-Geogrid Interaction Coefficient: 1.0
Percent Coverage of Geogrid: 100 Percent

External Loading
250 PSF traffic loading located 5 ft from back of wall

Inspection
Wall construction shall be periodically inspected under the direction of NGA.



Wall Height (feet)	Number of Geogrid Layers	Geogrid Length (feet)	Geogrid Height Above Leveling Pad/ Geogrid Type (feet)					
4	2	5.0	0.67 5XT*	2.67 5XT				
6	3	6.0	0.67 5XT	2.67 5XT	4.67 5XT			
8	4	7.0	0.67 5XT	2.67 5XT	4.67 5XT	6.67 5XT		
10	5	9.0	0.67 5XT	2.67 5XT	4.67 5XT	6.67 5XT	8.67 5XT	
12	6	11.0	0.67 5XT	2.67 5XT	4.67 5XT	6.67 5XT	8.67 5XT	10.67 5XT

*Mirafi 5XT Geogrid (or equivalent)

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NGA
GEOTECHNICAL ENGINEERS & GEOLOGISTS
17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax 481-2510
Snohomish County (425) 337-1669
Wenatchee/Chelan (509) 784-2756
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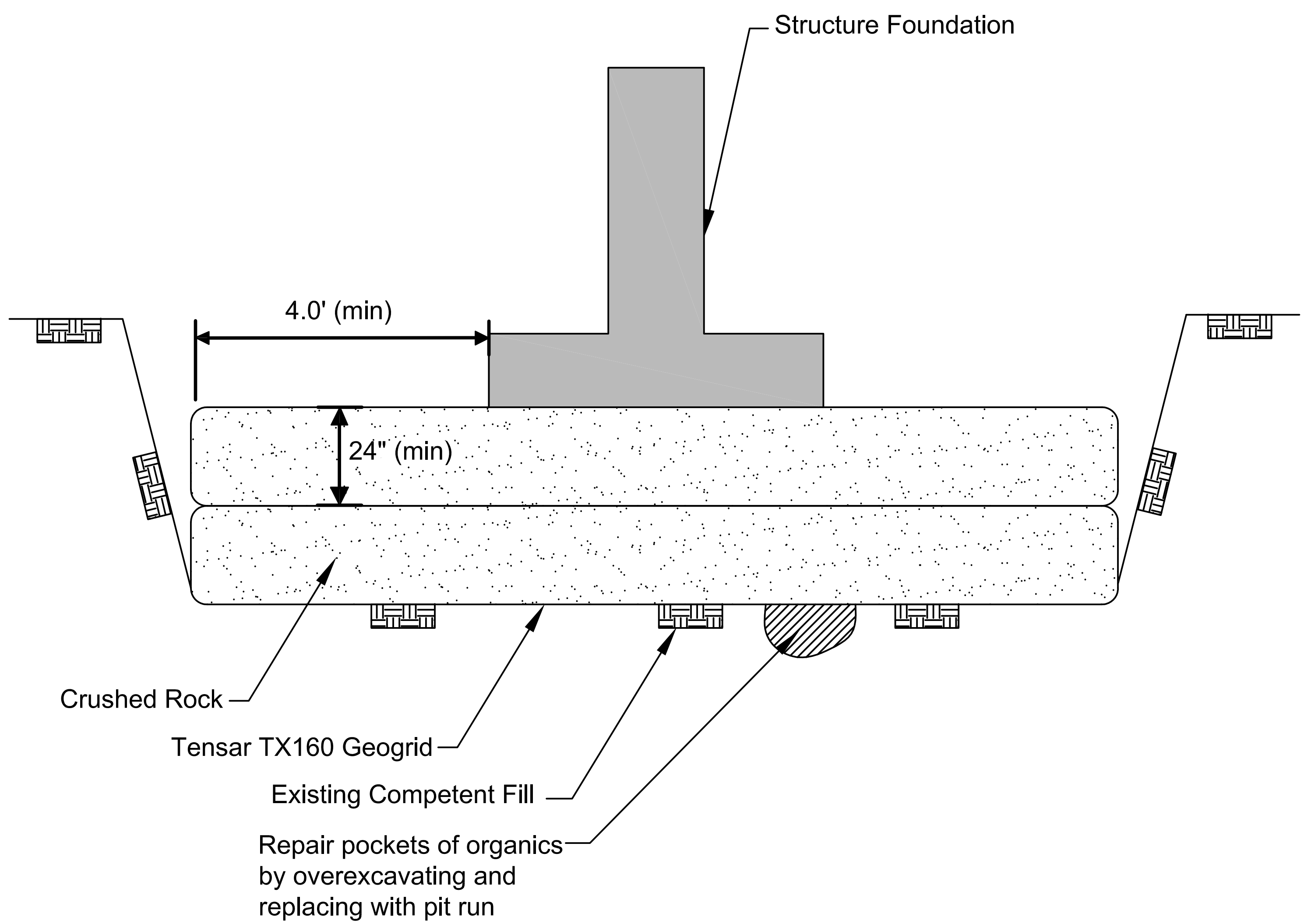
Union Hill Self-Storage
Typical Keystone Block
Reinforced-Wall Detail

Project Number
969616

Figure 20

Schematic Foundation Subgrade Treatment Detail

NOT TO SCALE



Project Number	Union Hill Self-Storage Schematic Foundation Subgrade Treatment Detail	Nelson Geotechnical Associates, Inc.
969616		Geotechnical Engineers & Geologists
Figure 21		

NGA
Nelson Geotechnical Associates, Inc.
Geotechnical Engineers & Geologists
17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax 481-2510
Snohomish County (425) 337-1669
Wenatchee/Chelan (509) 665-7696
www.nelsongeotech.com

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